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Finite element modelling of floodplain inundation

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Synopsis

Flood inundation phenomena typically occur over reach lengths of 5 - 30 km and incorporate a number of complex flow mechanisms. These include a momentum transfer between the main channel and floodplain and turbulent mixing caused by the delivery of water to the floodplain from the channel and its subsequent return. However, currently available one dimensional schemes applicable at scales appropriate to floodplain inundation processes cannot effectively simulate such processes. This is due to both an incomplete description of the flow physics and a failure to treat floodplain areas in realistic fashion. More complex two and three dimensional models, which have these capabilities, have only been applied over very short reach lengths (c. 0.5 - 2 km) and rarely to compound meandering channels.

This thesis reports on the further development of a generalized two dimensional, finite element code (RMA-2) to meet this research need. This is achieved via a series of modifications to the numerical model and to the physical representation by finite elements that enable river channel/floodplain flow at the long reach scale to be effectively simulated.

Evaluation of the enhanced RMA-2 scheme follows a three stage strategy. Firstly, the assumptions underlying the scheme are examined to identify possible inconsistencies. Secondly, tests are undertaken to assess whether the specified physical model has been correctly transferred into computer code. This is achieved via sensitivity analysis, examination of numerical stability issues and investigation of model response to abnormal parameterization. Thirdly, model predictions of flow field information are compared to observed field data in the context of an application of the enhanced model to an 11 km reach of the River Culm, Devon, UK.

Results from this evaluation process indicate that the enhanced RMA-2 model is capable of simulating main channel/floodplain momentum transfer and the two dimensional effects associated with compound meandering channels at this scale. Model simulations compare favourably to field data, both for specific cross sections and over the entire mesh.

Finally, extension of this core modelling capability is begun via the development of two model application scenarios. These demonstrate the likely utility of the enhanced scheme for the assessment of flood risk and the investigation of sediment deposition processes in floodplain systems.

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Memorandum

This thesis is the original work of the candidate except where acknowledgement is given and has not been submitted for a higher degree in this or any other University.

Paul Bates

Paul Bates.

August, 1992.

Summary of contents

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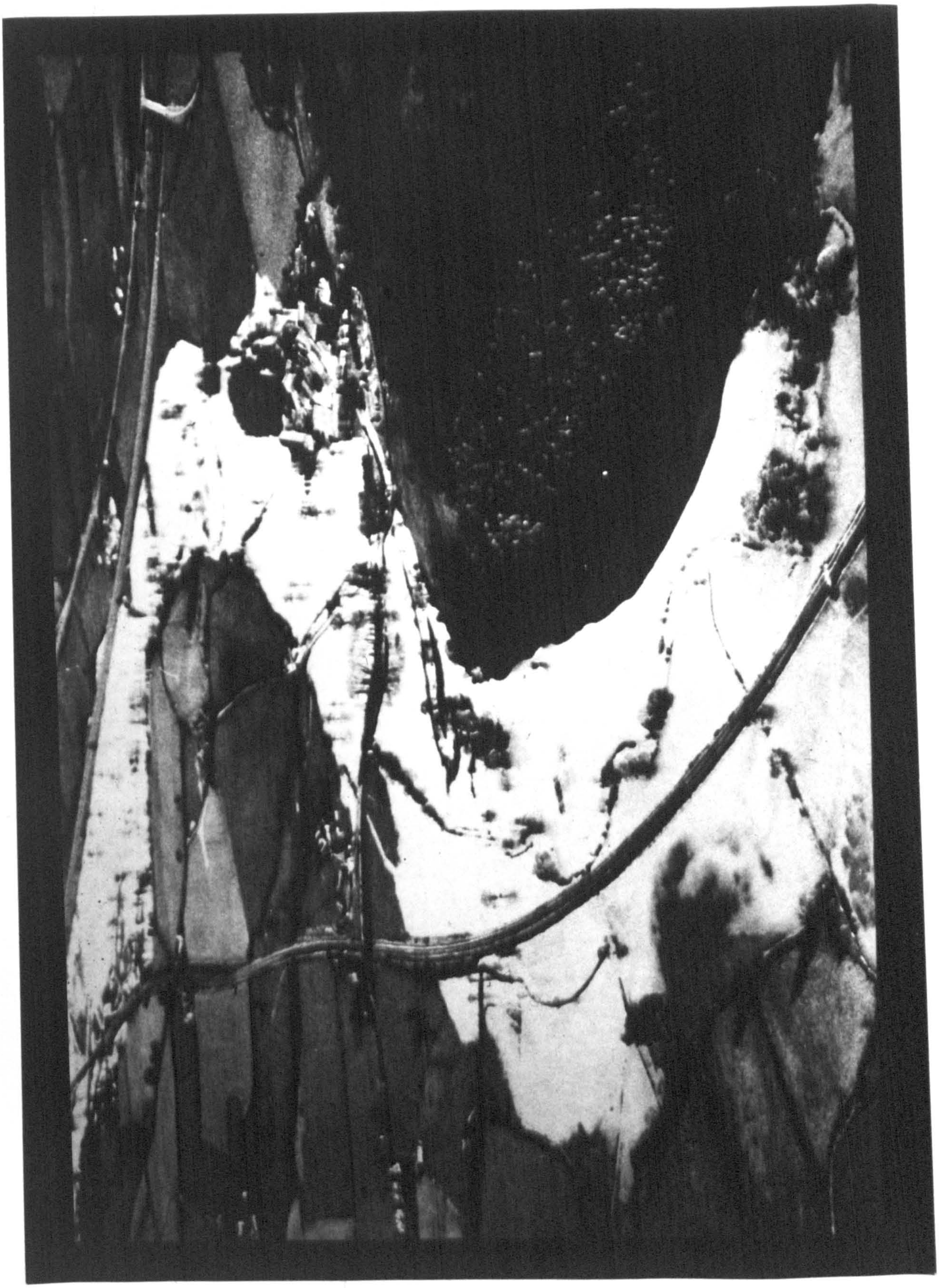
Symbol	*Description*	*Units*[1]
a,b,c	empirical coefficients from stage-discharge relationship	-
g	gravitational acceleration	ms^{-2}
h	depth of flow	m
h_c	channel depth	m
n	Manning coefficient	-
p	pressure	Pa
t	time	s
$\mathbf{u}$	unit velocity vector	$m\ s^{-1}$
u,v,w	velocity components	$m\ s^{-1}$
x,y,z	spatial coordinates	m
y_f	floodplain flow depth	m
z_o	bottom elevation	m
B_c	channel base width	m
C	Chezy coefficient	-
D/Dt	substantive derivative (rate of change following the fluid)	-
$\mathbf{F}$	set of problem specific terms incorporated into the Navier-Stokes force-momentum equations	-
F_i	model input	-
F_o	model output	-
Fr	Froude number	-
L	characteristic length scale of a flow	m
Q	discharge	$m^3\ s^{-1}$

1. S.I. units are used as standard throughout this report.

R	domain coefficient depth range	m
Re	Reynolds number	-
R_c	radius of curvature	rad
RSI	relative sensitivity index	-
S_f	sensitivity function of the factor f	-
U	characteristic velocity of a flow	m s^{-1}
US_f	unit sensitivity to the factor f	-
V_a	local wind velocity	m s^{-1}
Vf_1	flow velocity within meander belt	m s^{-1}
Vf_2	flow velocity outside meander belt	m s^{-1}
W_m	meander width	m
W_t	total width	m
ε	turbulent eddy viscosity coefficient	Pa s
ϕ	time interval for Crank-Nicholson procedure	-
λ	local latitude	rad
λ_m	meander wavelength	m
μ	viscosity	Pa s
ν	kinematic viscosity	m^2 s^{-1}
θ	empirical coefficient in Crank-Nicholson procedure	-
ρ	fluid density	kg m^{-3}
σ	domain coefficient	-
τ	fluid shear stress	N m^{-2}
τ_b	local boundary shear stress	N m-2
τ_o	mean boundary shear stress acting on the wetted perimeter	N m^{-2}
ω	rate of earths angular rotation	rad s^{-1}
ψ	local wind direction	rad
ζ	empirical coefficient in wind stress calculations	-
Γ	Galerkin residual	-

Σ	summation	-
∂	partial differential operator	-
Δ	change in a particular quantity	-
∇	gradient operator	-
$\nabla\cdot$	divergence operator	-
∇^2	Laplacian operator	-
f	Newton-Raphson error function	-



A 1 in 5 year recurrence interval inundation event (event **b** in Chapter 5) on the River Culm looking upstream from Silverton Mill. Note topographic complexities such as the meandering channel, mill races and change in floodplain orientation.

I Model identification and development

CHAPTER 1

Introduction

There is no currently existing capability to model flow in compound meandering channels at a high level of spatial and temporal resolution for reach scales appropriate to floodplain inundation (Gee *et al.*, 1990). Yet, such a modelling approach is essential for the realistic simulation of floodplain inundation processes. One dimensional flow models, the current standard analytic tool for such flow problems (Samuels, 1990), are unable to resolve the lateral flow field variations characteristic of inundated floodplains (Knight, 1989). Recent research initiatives, such as the SERC Flood Channel Facility programme (Knight and Sellin, 1987; Shiono and Knight, 1991), have indicated the hydraulic complexity of compound meandering channel flow, which includes such mechanisms as a lateral momentum transfer from main channel to floodplain flows (Rajaratnam and Ahmadi, 1981; Knight and Hamed, 1984; Ervine and Ellis, 1987; Prinos, 1989) and a delivery of water onto the floodplain and its subsequent return to the channel (Rajaratnam and Ahmadi, 1989; Kiely, 1990; Willetts and Hardwick, 1990). It is now appropriate to include such detail in mathematical model formulations to obtain a more realistic description of floodplain inundation processes. However, complex two and three dimensional hydraulic models, which have this capacity, have only been applied to reaches up to a few river widths (0.5 - 2 km) in length (see for example Niemeyer, 1979; Su *et al.*, 1980; Samuels, 1985; Akanbi and Katopodes, 1988) and not to the relatively long (10 - 30 km) reaches over which floodplain inundation typically occurs. This thesis will argue that to effectively simulate the spatial and temporal dynamics of floodplain inundation, hydraulic models must operate at a scale commensurate with the inundation process. This will therefore necessitate an **order of magnitude of increase** in the reach length which high spatial and temporal resolution hydraulic models can consider.

The development of such a modelling capability is, therefore, the central task of this thesis. The purpose of this Chapter is to demonstrate the wider justification for such a modelling approach and thereby isolate those criteria which any successful modelling solution should include. This will enable a set of specific research objectives to be established. Given the hydraulic complexity of open channel flows, fulfilling these objectives will necessarily involve assumptions being made concerning those flow processes which require inclusion in any mathematical model formulation. The process set for this problem type will be identified in Section 2.1 where the fluid mechanics of free surface flows in compound meandering channels are considered. Development of a modelling approach that includes these processes, and therefore provides the modelling capability outlined above, can then be undertaken. An appropriate modelling strategy for this task will be identified in Section 2.3. Chapter 3 will then describe the selection of a prototype model of this class and its further enhancement to enable long reach river channel/floodplain applications to be considered. Following this, a research design for testing and application of the enhanced model will be defined in Section 3.3, which subsequent Chapters of this thesis will pursue.

1.1 Wider needs for flood inundation modelling

In addition to meeting the research needs identified above, a high spatial and temporal resolution model of floodplain inundation over reach scales of 10 - 30 km has significant application potential in a number of fields. These range from meeting the information needs for planning and management of practising water engineers to the investigation sediment transport problems. Potential applications are considered here under the three broad categories of engineering, geomorphological, and hydrological perspectives on floodplain inundation modelling and will provide substantial justification for undertaking the modelling programme outlined above.

1.1.1 Engineering perspectives

Floodplain environments are under increasing pressure from developers (Brookes, 1988). In certain highly populated areas, such as South Eastern England, floodplains may represent the only available building land. In order to undertake such tasks as impact assessment of proposed developments, the planning of flood alleviation schemes (Johnson and Capel-Davies, 1990) and determination of the standard of flood protection along particular river reaches (Bartlett and Townsend, 1990), environmental managers and water engineers have an increasing requirement for model generated information (see for example Gardiner, 1990). The number of river reaches where it is necessary to implement complex hydraulic models has therefore increased. However, data and calibration needs for currently available model formulations are often extensive, requiring recourse to data collection programmes in order to overcome such problems. These substantially increase the time and resources required to complete any modelling study. The number of reaches where current model formulations can be economically applied is therefore limited. Furthermore, a shift in demand towards more sophisticated flow modelling techniques for a significant number of engineering and planning problems has also become evident. There are two main reasons for this:

i) Recent advances in our understanding of flood flow hydraulics (see for example Knight, 1989; Falconer, *et al.*, 1989; White, 1990) has led to the identification of certain deficiencies in current modelling approaches. For example, currently available one dimensional flow models such as HEC-1 (HEC, 1981) and FLUCOMP (Samuels, 1983a) are unable to adequately resolve the momentum exchange mechanism now known to be significant for out-of-bank flow in compound meandering channels.

ii) The implementation of sophisticated river corridor management programmes has increased the scope and variety of predictive products required from modelling studies. For example, the Standards of Service Approach (Jaeggi and Zarn, 1990) requires the identification of the inundated areas associated with particular event frequencies, for each reach under consideration. For such planning purposes river corridors are typically divided into unit reaches of the order of 7 - 14 km. The prediction of such complex quantities necessarily requires the development of sophisticated modelling techniques that are applicable at such scales.

This shift has further increased the need to revise existing hydraulic modelling capabilities.

The above arguments demonstrate that to effectively respond to the needs of environmental managers and water engineers, new hydraulic models are required that can not only generate accurate hydrographs for particular cross sections, but also:

i) are able to provide a range of predictive products, including water depths, velocities and inundated areas over entire reaches,

ii) can be applied to a wide range of environments at reach lengths appropriate to the complete documentation of floodplain inundation,

iii) can be parameterized on the basis of extremely sparse data sets,

iv) require only minimal calibration.

At present numerical models with these capabilities do not exist.

The development of the particular flow model proposed in this thesis would seem to have considerable potential for meeting these requirements. A high spatial and temporal resolution, and therefore more complete, description of the flow field not only allows a greater variety of products to be generated but also reduces the reliance on artificial manipulation of model parameters to obtain a realistic simulation of a given flow field. The above discussion also indicates a number of criteria that should be incorporated into model design, such as a reduction in data requirements. Such developments will lead to a modelling scheme for which the number of reaches where applications are numerically stable, and can be economically justified, is maximised.

1.1.2 Geomorphological perspectives

A range of geomorphological processes, from localised bank collapse (Thorne and Tovey, 1982; Thorne, 1992) to long term floodplain evolution (Lewin, 1978), occur within river channel/floodplain systems. Although these operate over many space and time scales, the power to do all types of geomorphological work in this system is ultimately provided by water flows within the river channel/floodplain corridor. It follows that an understanding of these flow processes is an essential prerequisite for a variety of geomorphological investigations. Recently, a shift in the spatial and temporal scale of geomorphological investigation, relative to more traditional long term perspectives, has introduced major new requirements for background data (Bates *et al.*, 1992) and thereby increased the detail and complexity of the flow predictions required. For example, the development of equipment for the continuous monitoring of river bank erosion (Lawler, 1992) needs to be matched by high resolution hydraulic models, capable of generating flow depths and velocity vectors at similar time and space scales, in order to permit meaningful data analysis. The development of caesium^{137} dating techniques for floodplain sediment cores (Walling *et al.*, 1986; Walling and Bradley, 1989; Walling *et al.*, 1992) provides a further example of the way in which advances in data collection techniques have created new modelling requirements. This technique allows average floodplain deposition and erosion rates for the last 35 years to be documented for a large number of floodplain sites. It is impractical, however, to collect flow measurements at a commensurate scale (horizontal ranges of 10 -100 m) over a sufficiently wide range of event magnitudes to effectively interpret this data. In the light of this, there is a need to develop models capable of simulating the behaviour of floodplain flows to a high degree of spatial and temporal resolution, in order to reduce direct reliance on field measurement.

In order to effectively contribute to geomorphological studies, it would clearly be of benefit if high resolution flow modelling schemes could also:

i) Incorporate complex topography into their numerical discretization. Floodplain microtopographic variations have been identified (Walling *et al.*, 1986) as one of the primary controls on the spatial heterogeneity of floodplain sediment transport processes. Modelling schemes that attempt to investigate such processes must, therefore, be capable of accounting for this complexity.

ii) Be capable of dynamic simulations. Temporal variations in geomorphic processes during the course of flood events, such as the deposition and reworking of overbank sediments, are potentially highly significant. However, little data concerning such processes exists. In the case of floodplain sedimentation, data is only available from the post event analysis of deposits on the floodplain (see for example Mansikkaniemi, 1985; Lambert and Walling, 1987). This gives no indication of intra-event process variation. The development of geomorphologic models driven from a dynamic flow simulation would, for the first time, allow the importance of this high resolution temporal variation to be assessed over the duration of whole events.

iii) Be capable of simulating sequences of flood events. The geomorphic effectiveness of water flow events may be as much due to their sequence as their absolute magnitude (Anderson and Calver, 1977; Newson, 1980). However, current models of river sediment dynamics (Ponce *et al.*, 1979; Borah *et al.*, 1980; Lyn, 1987), which have been incorporated into distributed models (e.g. Storm *et al.*, 1987), concentrate on specific short time scale geomorphological processes (Beven and Carling, 1989) rather than the dynamic flow processes driving such phenomena. These, therefore, cannot readily be used to simulate sequences of events of varying magnitudes. The development of the flood event simulator proposed in this Chapter would allow such sequences to be modelled and would potentially mean that the hydraulic conditions for every geomorphologically significant event in a particular flow record could be obtained.

We may conclude from this discussion that the modelling developments proposed in this Chapter are supported by a consideration of current geomorphological research needs. These range from requirements for background data to the fulfilment of specific methodological objectives. Moreover, a number of additional model design criteria have also been identified.

1.1.3 Hydrological perspectives

Over the last decade the development of process representation in physically based distributed hydrological models, such as the SHE (Abbott et al., 1986; Bathurst, 1986) and the IHDM (Beven et al., 1987; Calver, 1988), has begun to allow simulation of distributed surface and subsurface runoff production within catchments (Beven and Carling, 1989). However, recent debate (see for example Beven, 1987; Beven, 1989; Anderson and Burt, 1990) has questioned the validity of this 'internal' predictive ability. Anderson and Burt (1990) report the application of two physically based distributed schemes, SHE and VSAS (Bernier, 1985; Troendle, 1985), to the Bicknoller Coombe catchment, Somerset, UK, for which detailed tensiometric and other subsurface hydrological data are available for 'internal' validation. Despite relatively complete knowledge concerning subsurface flow paths and surface contributing areas, both models failed to adequately describe hydrological process variation within the catchment, particularly for channel or near channel areas. This is significant given the concentration of hydrological activity in such regions, which may include the origin and development of variable source areas during storm events, the operation of solute buffer zones in near channel areas and dynamic changes in process rates as a result of episodic inundation. Reasons for this failure include:

i) Insufficiently detailed discretization of topography. Grid scales for the application of physically based distributed hydrological models are typically of the order of 100 - 2000 m. However, even the 50 m discretization used for the Bicknoller Coombe study (Whitelaw, 1988) may be too coarse to adequately represent flow processes, particularly in light of the previous argument concerning the importance of microtopographic variations on the floodplain flow field.

ii) The difficulty of establishing effective parameter values at the grid scale (Beven, 1989; Binley *et al.*, 1989). The governing equations for physically based distributed hydrological models have typically been derived from laboratory experiments concerning small scale homogeneous systems. Such models therefore assume that each grid square is an homogeneous unit and that the small scale physics on which such models are based apply at the grid scale. In reality, hydrological processes within grid squares show considerable heterogeneity, which may be at least as significant as variations between grid squares. Physically based distributed models cannot account for this heterogeneity and merely represent these processes as a single lumped parameter value which may be difficult to establish from field data.

iii) Problems with model calibration procedures. A large number of model parameters (Beven, 1989) and inherent uncertainties in parameter estimation (Rogers and Anderson, 1987; Binley et al., 1991) make calibration of physical based distributed hydrological models a complex task. In an application of the SHE model to the Wye catchment at Plynlimon (Bathurst, 1986) calibration checks against internal state variables showed significant errors. Furthermore, this study found that it was impossible to devise a calibration scheme capable of maximising predictive ability for all process observation sites.

Thus in physically based hydrological models there is an evident lack of attention to two specific areas; in the representation of channel and near channel areas and in the construction of models which are able to accurately simulate 'internal' process behaviour. This thesis will argue that instead of attempting to represent processes across the entire catchment, modelling effort should concentrate on the zones of most intense hydrological activity, such as floodplains. The development of internally consistent models of floodplains areas will then provide a core capability to which other hydrologic models, such as distributed hillslope flow schemes, can be coupled. Moreover, this discussion again highlights the necessity to include topographic complexity into floodplain flow model formulations and reduce the reliance on extensive calibration procedures.

1.2 Research objectives for floodplain inundation modelling

In addition to confirming a substantial wider justification for floodplain inundation modelling, the preceding discussion has identified a number of design criteria for inclusion in high spatial and temporal resolution models of river channel/floodplain flow for long reach scales. The scheme must:

i) have parsimonious requirements for data,

ii) be capable of simulating water depths, velocities and inundation over horizontal scales of 10 - 100 m,

iii) be capable of dynamic simulations,

iv) be able to account for complex topography.

These, therefore, constitute the research objectives which the model developed in this thesis will aim to fulfil. This task begins in Chapter 2 with the identification of a prototype modelling strategy which can be further developed to meet the research needs outlined here.

CHAPTER 2

Modelling flood inundation

A need has been identified in Chapter 1 to attempt the modelling of flow in compound meandering channels at a high level of spatial and temporal resolution for reach scales appropriate to floodplain inundation. This is the primary research objective of this thesis. Basic design criteria for such a modelling scheme have also been delimited. It was concluded that the scheme must:

i) have parsimonious requirements for data,

ii) be capable of simulating water depths, velocities and areas of inundation over scales of 10 - 100 m,

iii) be capable of dynamic simulations,

iv) be able to account for complex topography.

This Chapter identifies a modelling strategy capable of satisfying these criteria. As an initial step, the hydraulics of open channel flows are examined to determine the specific set of processes which model formulations need to include to achieve the above objectives. It will then be possible to a select an appropriate simulation strategy which represents these processes and can act as a suitable vehicle for this project. Finally, those developments to this strategy necessary to enable the high resolution simulation of river channel/floodplain flow at a long reach scale will be considered.

2.1 Flow in meandering compound channels - the physical processes

2.1.1 General principles of fluid motion in open channels

Free-surface flows in open channels may be classified in terms of changes of depth or velocity with respect to both time and space (French, 1986). During the passage of a flood wave open channel flow will vary in both these respects, being unsteady in time and non-uniform in space. Steady, uniform flow is, however, often assumed to apply locally to allow uniform flow formulae, such as the Manning equation, to be applied (Morisawa, 1985).

In addition, flow may be described in terms of two non-dimensional numbers, the Reynolds number and the Froude number, which represent thresholds of change in the hydraulic system. The Reynolds number, which depends on the relative magnitude of viscous and inertial forces within the fluid, differentiates between laminar and turbulent flow and is calculated by:

$$Re = \frac{UL}{\nu} \tag{2.1}$$

where Re = Reynolds number,
U = characteristic velocity of the flow,
L = characteristic length scale of the flow,
ν = kinematic viscosity.

For laminar flow (Re <500) viscous forces dominate and streamlines in the flow are parallel and smooth with water apparently 'sliding' in layers. When inertial forces increase relative to viscous forces (Re >2000) the flow becomes turbulent. Here streamlines are not parallel, chaotic velocity fluctuations occur and exchange of turbulent energy causes vigorous mixing within the fluid body. Between these two states (500< Re <2000) flow is deemed to be transitional, exhibiting characteristics of each type.

The Froude number depends on the ratio between inertial and gravity forces and is used to distinguish between sub- and supercritical flow regimes. It is calculated by:

$$Fr = \frac{U}{\sqrt{gL}} \tag{2.2}$$

where Fr = Froude number,
g = gravitational acceleration.

If this ratio is less than 1 the flow is termed subcritical or tranquil. In this situation gravity waves can propagate upstream. If the Froude number is greater than 1 then the flow is considered to be supercritical, disturbances can only propagate in a downstream direction and surface waves may become unstable and break. This causes an energy loss which generally acts to increase the flow resistance (Petts and Foster, 1985). Rapid transitions between these two states commonly occur in natural open channel systems. In particular a transfer of flow from super- to subcritical flow is associated with a stationary surge wave termed a hydraulic jump. This transition is rapid, involving a large energy loss due to turbulence (Chadwick and Morfett, 1986) and usually occurs where the form of the channel bed changes suddenly. **Under these criteria flow in natural open channels can be seen to be unsteady, non-uniform and turbulent at almost all times, with frequent transitions between sub- and supercritical states.**

Other processes relevant to this study which have a dynamic impact on the velocity field in open channels include frictional and shear forces. These will now be discussed in detail.

Flow passing over a fixed boundary generates frictional resistance, retarding flow in the vicinity of the bed. This frictional force represents the sum of a number of effects which Knight (1989) states can be classified according to the nature of the channel boundary. These categories are:

i) Rigid boundary - here resistance is determined by three factors; the shape of the cross-section, the non-uniformity of the textural roughness around the wetted perimeter and the form drag arising from three-dimensional effects.

ii) Loose boundary - fluid flowing over a mobile boundary can interact with the bed. The bed is thus no longer plane but develops through a particular morphology which introduces additional resistances.

iii) Flexible boundary - due to vegetal growth. Flow resistance due to vegetation has been shown to have a complex relationship with velocity, deflected roughness height, vegetation density and vegetation stiffness (Kouwen and Li, 1980). Floating debris can also lead to the blocking of channels (Klassen and Zwaard, 1974) and contribute to flow resistance.

It is obvious from these studies that boundary friction in open channels is not constant but varies dynamically, interacting with the flow in a highly complex manner.

This resistance to flow results in the formation of a boundary layer at the channel bed, within which a gradient of average velocity occurs. This is taken to be approximately logarithmic in turbulent flow and will extend to the surface in most channel flow situations (Richards, 1982). In addition to this time-averaged pattern of velocity, vortices in turbulent flow cause fluctuations in the instantaneous velocities and superimpose secondary motions on the primary downslope flow component. This pattern is further complicated by heterogeneities in cross-sectional geometry and boundary friction. The impact of bed and bank roughness heterogeneity on flow structure has been studied by Naot (1984). He demonstrated that the dominant presence of the strong vortex induced by the free surface restricted the majority of these effects to the lower part of the channel, where there were significant changes in the turbulence structure. The structure of the secondary currents induced was also shown to depend on the spatial distribution of roughness elements. The pattern of secondary currents is therefore complex and plays an important role in such phenomena as energy dissipation in turbulent flows (Morisawa, 1985) and the initiation of sediment transport (Richards, 1982), as well as giving rise to an additional viscous resistance within the fluid body (Petts and Foster, 1985). In addition to turbulent fluctuations, distinct secondary flow structures can also form in open channels. For example, flow asymmetries set up by channel curvature can create rotating secondary cells (Bathurst *et al.*, 1979; Odgaard and Bergs, 1988). Such flow patterns have been shown to have a significant impact on channel planform and cross-sectional development (see for example Ikeda *et al.*, 1981), implying feedback effects on the flow structure itself over medium to long time scales. **The net result of these effects is therefore the production of a highly three-dimensional velocity field (see figure 2.1).**

The shear stress, defined by the momentum gradient and the degree of molecular energy exchange, can be expressed as:

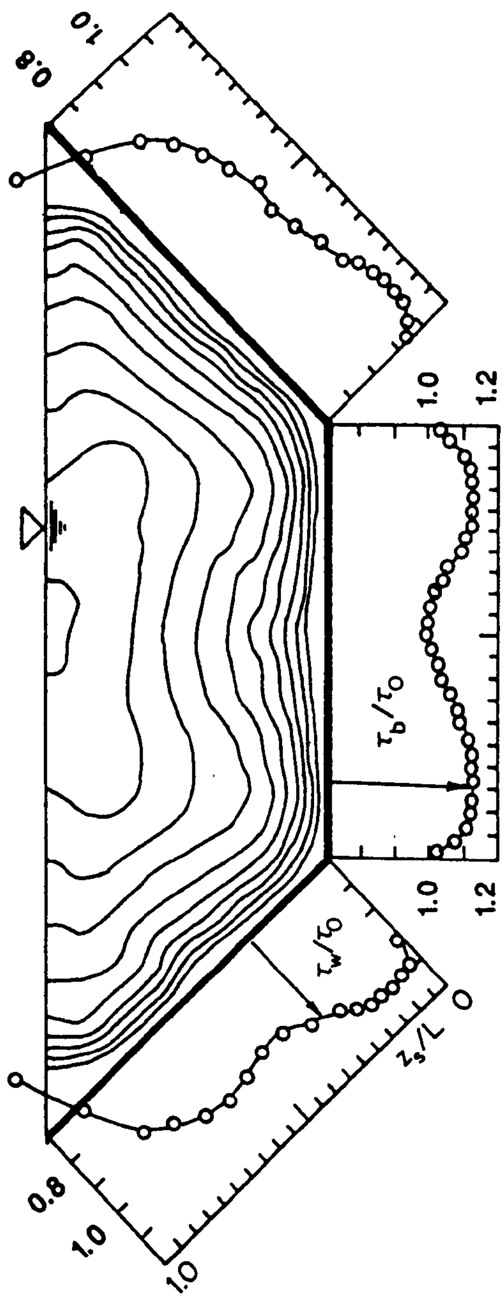


Figure 2.1: An example of the three dimensional velocity field in open channel flow and its relationship to dimensionless boundary shear stress, τ_b/τ_o, (taken from Knight, 1989)

$$\tau = \rho \mu \frac{dv}{dy} \tag{2.3}$$

where τ = shear stress at any height above the bed,
ρ = fluid density,
μ = viscosity,
dv/dy = rate of change of velocity with vertical distance.

Boundary shear stress will thus be dependent on the velocity gradient close to the bed (Bathurst *et al.*, 1979). Peak values of boundary shear stress will occur in regions of downwelling where the isovels are compressed. This is clearly demonstrated by Figure 2.1, which shows non-dimensional boundary shear stresses generated by the velocity field in a trapezoidal prismatic channel (taken from Knight, 1989). The three-dimensional velocity field therefore introduces complexity into the shear and boundary shear stress fields in open channels (see for example Nouh and Townsend, 1979; Chiu and Hsiung, 1981; Das and Townsend, 1981; Knight, 1981; Knight *et al.*, 1982; Knight and Patel, 1985). The spatial distribution of boundary friction elements has also been shown to have a significant impact on shear stress distributions, with wide variations being shown (Ghosh and Jena, 1971; Ghosh, 1972; Ghosh, 1973; Ghosh and Mehta, 1974). In addition, boundary shear stress is an important parameter in many sediment transport phenomena (see for example Parsons, 1960; Knight, 1989), which can interact with the flow structure via feedback mechanisms (Yalin and Finlayson, 1972). For example, bedform development can occur in response to secondary currents (Willetts and Hardwick, 1990) and this can then influence the flow structure. It has also been suggested (Odgaard, 1984) that, in certain circumstances, shear forces can act to induce secondary flows in open channels, although the physical mechanism generating this effect is not clear.

The above discussion highlights the fact that channel geometry, boundary friction, shear stress, primary and secondary flows and sediment concentration in open channels all vary three-dimensionally and interact dynamically with one another (Chiu and Hsiung, 1981). This set of physical processes is, however, further complicated for channel types more frequently encountered in the consideration of flood flows. The complexity of this flow field is therefore such that mathematical models of open channel flow must be based on a series of simplifying assumptions. These serve to isolate those processes most significant to the particular problem under consideration. In order to undertake the dynamic simulation of compound meandering channel flow for reach scales of 10 - 30 km the mathematical model developed in this thesis must be capable of simulating the bulk flow characteristics of unsteady, non-uniform fluids and incorporate the effects of spatially heterogeneous boundary friction and turbulence. Simulation of secondary currents is not, however, relevant to this flow problem as these operate at small time and space scales that cannot be adequately resolved in long reach models.

2.1.2 Physical flow processes in compound channels

Natural channels are usually compound in cross section, consisting of a main channel, which carries flow at most times, and one or two floodplains which are inundated when the bankfull stage of the main channel is exceeded (see Figure 2.2). These floodplains may act as either temporary stores of stationary water or as part of a compound channel conveying water downstream. In this situation the flow structure will be further complicated by a more variable cross-sectional geometry, heterogeneous boundary roughness and a number of additional processes specific to compound channels. A large body of relevant literature now exists (Holinrake, 1987) based on both field and laboratory studies. However, most of this work has concentrated on processes in straight channels rather than the more complex meandering case.

2.1.2.1 Straight compound channels

At low floodplain depths in compound channels a shear layer has been observed to develop (Sellin, 1964) between slow flowing floodplain water and faster moving water in the main channel. This interaction has been found to be highly significant, acting to further complicate the three-dimensional flow structures present in open channel flow (Knight, 1989). At higher flow depths, however, the two-stage channel begins to act as a single unit and the shear layer is damped out.

In terms of flow structure, the shear layer has been observed to induce a series of vortices, superimposed on the bed generated turbulence, with vertical axes along the channel/floodplain interface (Sellin, 1964; Zheleznyakov, 1971). Rajaratnam and Ahmadi (1981) have correlated the characteristic length scales of this interaction with the ratio of flow depths in the main channel and floodplain and found a positive linear relationship. A small number of studies have also investigated the three-dimensional

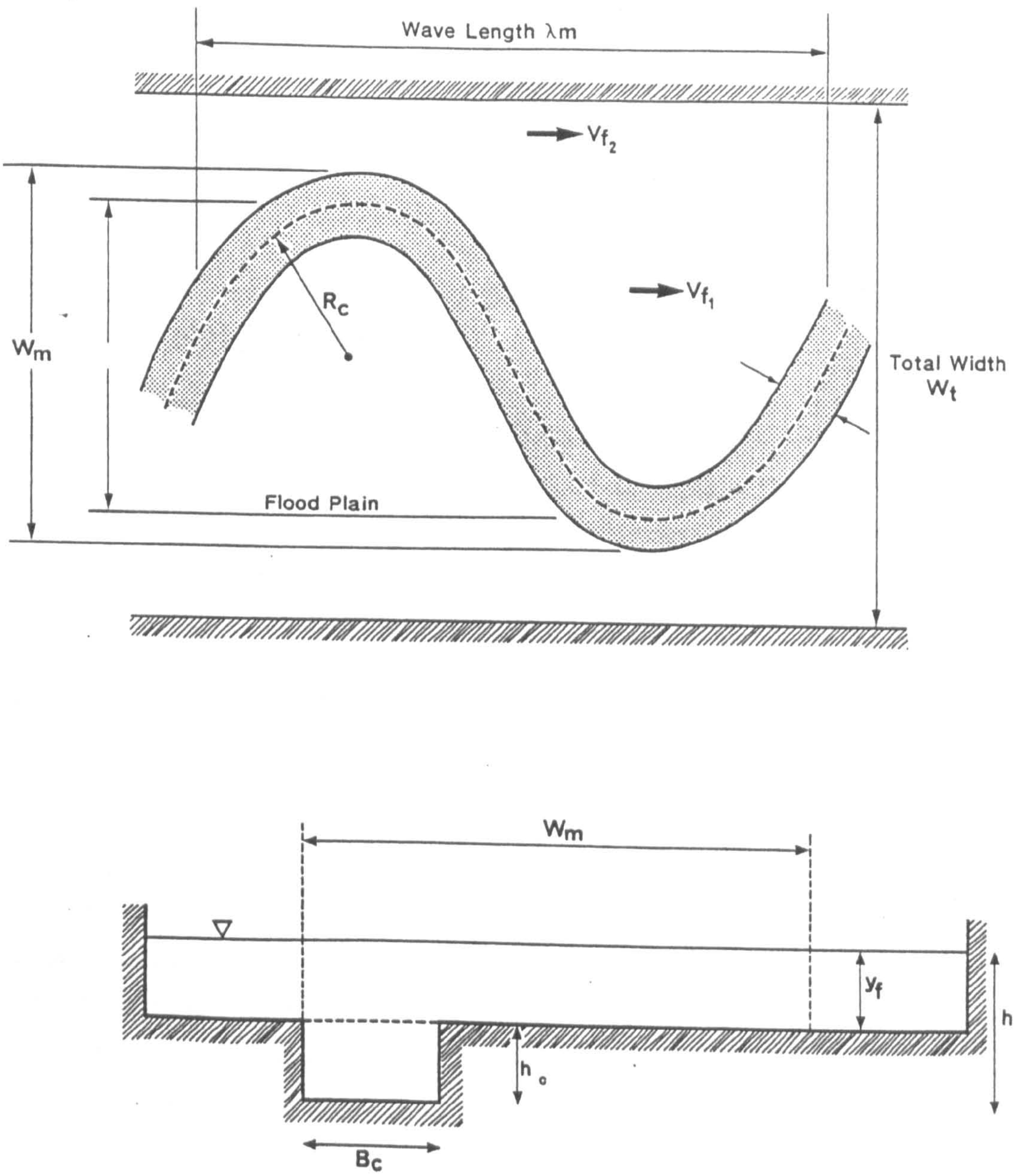


Figure 2.2: Definition of two stage geometry in plan and cross section, where; h_c = channel depth; h = total flow depth; B_c = channel base width; W_m = meander width; W_t = total width; R_c = radius of curvature; Vf_1 = flow velocity within meander belt; Vf_2 = flow velocity outside meander belt; v_f = floodplain flow depth; λ_m = meander wavelength.

nature of flow in this region. Prinos (1989) found higher turbulence intensities in regions of interaction, while Imamoto and Ishigaki (1989) used flow visualisation techniques to determine the structure of the shear layer in experimental channels. They observed an intermittent upwelling flow originating from the floodplain edge and inducing longitudinal secondary flow cells. The shear layer thus has a complex structure which conditions and influences flow processes in open, compound channels (see Figure 2.3).

The turbulent eddies in the shear layer operate as a lateral momentum transfer mechanism. This has the effect of retarding flow velocities in the main channel while increasing those on the floodplain (Knight, 1989; Prinos, 1989). Consequently, uniform flow formulae, which do not consider such interactions, either under- or overestimate net discharge (Bhowmik and Demissie, 1982; Ervine and Ellis, 1987; Shiono and Knight, 1991).

Bed shear stresses are also affected by the momentum exchange mechanism due to the velocity differential between main channel and floodplain flows (Radojkovic, 1976). The pattern of variation shown is essentially similar to that of flow velocity (Ghosh and Jena, 1971). Myers and Elsaway (1975) found that shear stresses in a straight compound flume were decreased in the main channel by up to 22% and increased on the floodplain by up to 260%, with shear ratios being significantly affected at low flow depths. These changes in shear stress have been described in terms of hydraulic parameters (Rajaratnam and Ahmadi, 1981) as well channel geometry (Knight *et al.*, 1983; Knight and Hamed, 1984; Holden and James, 1989).

More recent research using the SERC Flood Channel Facility (Knight and Sellin, 1987) has allowed the spatial distribution of both internal shear and Reynolds stresses to be examined (Shiono and Knight, 1989; Shiono and Knight, 1991). The Reynolds stress is the time averaged internal shear stress of a turbulent fluid on a particular plane (Massey, 1983), and is defined (for the x-z plane) by:

$$\bar{\tau}_{xz} = -\rho \; \overline{(u'v')} \tag{2.4}$$

$\bar{\tau}_{xz}$ = Reynolds stress in the x-z plane,

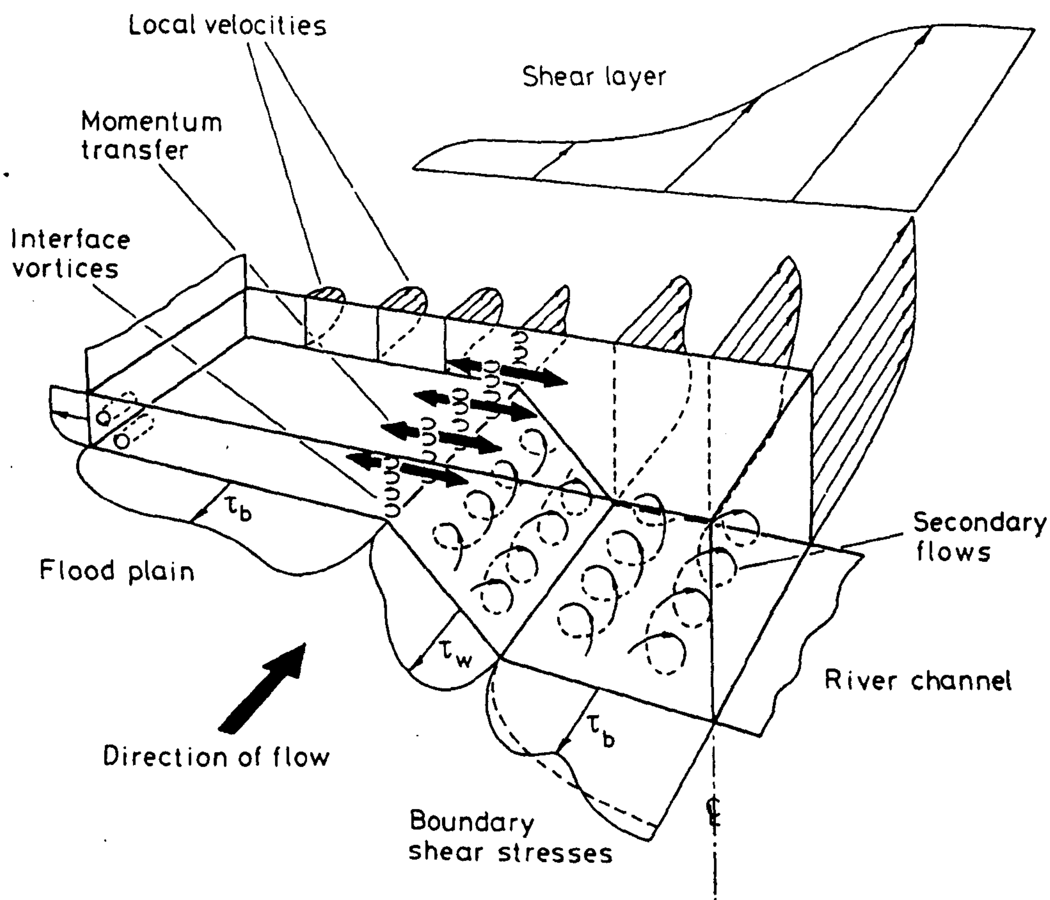


Figure 2.3: Hydraulics of out-of-bank flow in a straight compound channel (taken from Knight, 1989).

$\bar{u}'$ = time averaged instantaneous velocity in the x plane,

$\bar{v}'$ = time averaged instantaneous velocity in the y plane,

ρ = fluid density.

The distribution of Reynolds stresses thus gives an indication of the turbulence structure of a fluid. This research demonstrated that the Reynolds stresses in compound channels were highly non-linear in distribution, particularly in the regions of strongest lateral shear. This would therefore indicate a complex pattern of turbulence in these regions.

The momentum exchange mechanism is thus highly significant in terms of the flow structure irr compound channels. It is therefore essential that mathematical model formulations are able to simulate this lateral flow variation.

2.1.2.2 Meandering compound channels

In spite of the large body of research concerning flow processes in straight compound channels, significantly less work has focussed on the more complicated meandering case. This is surprising as meandering channels are typical of a large number of natural river systems and it cannot be assumed that the data derived for straight compound channels will be applicable in this instance.

Early work by the US Army Corps of Engineers (1956) and Toobes and Sooky (1967) found that floodplain flow intensified the secondary circulation in meandering channels. Smith (1978) demonstrated that under certain circumstances the meandering channel adds to the resistance of the floodplain flow. Physical and numerical model studies have allowed these general properties of overbank flow in meandering channels to be further examined (see Figure 2.4). For example, Rajaratnam and Ahmadi (1989) examined results from an experimental meandering channel which confirmed the results of Toobes and Sooky. They also demonstrated that procedures for modelling momentum exchange effects based on data from straight channels could not be directly applied to meandering channels due to high velocity channel flow spilling onto and running over the floodplain. Ervine and Ellis (1987) have tried to take account of these effects in a model for stage-discharge relationships in meandering channels with overbank flow. This was achieved by

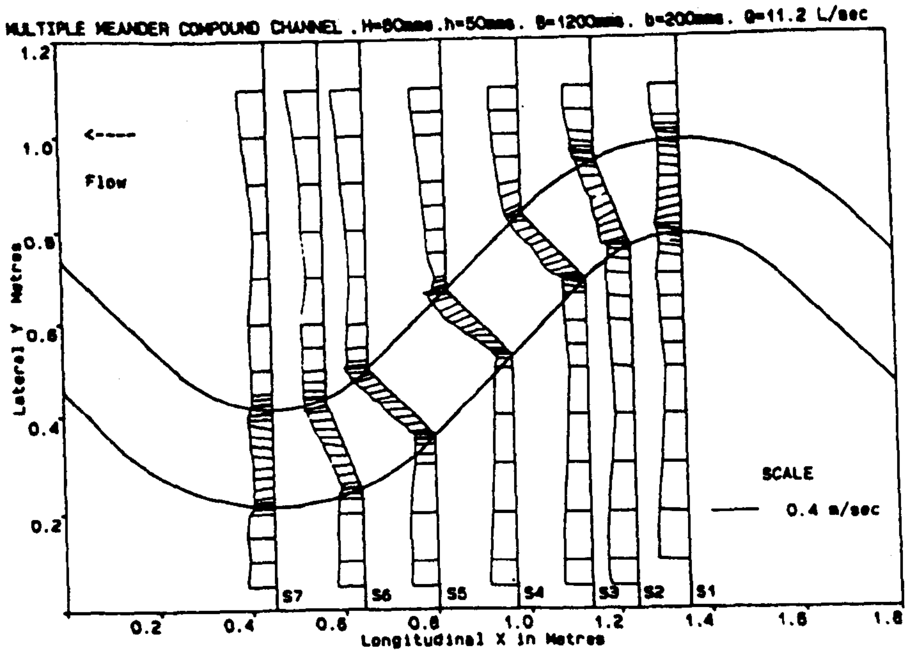


Figure 2.4: Experimental data from a physical model of a meandering compound channel showing lateral velocity vectors for a series of cross sections (Kiely, 1990).

estimating energy losses within and outside the meander belt. Initial results from this model showed that large increases in sinuosity could account for reductions in discharge of the order of 20-30%. This is of a similar order of magnitude to the momentum exchange mechanism outlined in Section 2.1.2.1. Further research (Willetts and Hardwick, 1990) in this area has even suggested that the spillage onto the floodplain of water delivered by the channel and its eventual return is a more vigorous momentum exchange mechanism than turbulent eddies at the floodplain/channel interface.

Attempts have also been made to disaggregate the physical processes occurring in meandering compound channels in order to explain the above effects. Using experimental flume data, Kiely (1990) has identified four flow mechanisms present in meandering compound channels additional to those recognised for straight compound channel situations. These are:

i) Secondary currents - such circulations are intensified in meandering channels.

ii) Horizontal shearing - as the floodplain flow impinges on channel flow a horizontal shear layer develops between the upper and lower parts of the channel.

iii) Flow expansion and contraction - longitudinal floodplain flow encounters a sudden drop when entering the main channel, leading to flow expansion. Similarly, as this longitudinal flow rises to re-enter the floodplain a contraction loss occurs.

iv) Downstream effects of cross over flow - a dip was found to occur in the velocity profile midway between the main channel and the edge of the floodplain. It was found that highly turbulent flow leaving the downstream side of cross-over sections was slowed to produce a low velocity filament between fast flowing water in the outer floodplain and the fast filament adjacent to the inner channel bend.

Such effects are largely unaccounted for in currently available hydraulic model formulations (Gee *et al.*, 1990). Certain models, such as EMBER (Samuels, 1983a), are able to simulate embanked rivers, however simulation of river channel/floodplain flow as a single continuum has only been attempted for a very few studies (see for example Samuels, 1985) and then never at reach scales appropriate to floodplain inundation processes. Yet, the above discussion indicates that inclusion of such effects is essential for the accurate simulation of such flow problems. In particular Willetts and Hardwick (1990) identify the delivery of water onto the floodplain and its subsequent return to the main channel as the most significant mechanism present in meandering compound channel flow.

2.1.3 Summary

A summary of the most significant recent advances in the investigation of river channel/floodplain hydraulics is provided in Table 2.1. This highlights the set of processes that require inclusion in mathematical model formulations in order to meet the modelling objectives outlined in Chapter 1 and the technical developments that now make such research possible. For reference, Table 2.1 also shows the parallel series of geomorphological developments identified in Section 1.1.2 that provide one aspect of the wider justification for this modelling approach. Thus, it has become apparent over the last decade that floodplains do not act simply as temporary stores of water, but rather, are a dynamic, three dimensional flow environment with a distinct set of flow process. Thus to effectively model the bulk characteristics of floodplain hydraulics at the long reach scale models are required that can simulate unsteady, non-uniform flow and incorporate spatially heterogeneous turbulence and boundary friction effects. In addition, two specific processes have been identified that must be included in any model formulation. These are:

i) the momentum exchange mechanism between main channel and floodplain flows,

ii) the delivery of water onto the floodplain and its subsequent return to the channel.

It is now appropriate to examine current modelling strategies for compound channel flow in order to identify a modelling strategy that can represent these processes.

Paper	Hydraulic advances	Geomorphological advances
Sellin (1964)	Identification of momentum exchange mechanism in compound channels.	
Toobes and Sooky (1967)	Identification of significant flow effects in meandering channels.	
Lewin (1978)		Investigation of long term floodplain evolution.
Ponce *et al.* (1979)		Development of short term models of sediment transport. Move towards high time and space scale investigation of geomorphological processes.
Rajaratnam and Ahmadi (1981)	Quantification of momentum exchange mechanism effects. Need to incorporate momentum exchange in hydraulic models established.	
Thorne and Tovey (1982)		Investigation of localised bank collapse processes.
Mansikkaniemi (1985)		Measurement of short time scale floodplain deposition rates.
Walling *et al* (1986)		Development of Cs-137 dating for floodplain sediment cores.
Sellin and Knight (1987)	Establishment of SERC Flood Channel Facility to investigate compound channel flow and provide data for new hydraulic models.	
Kiely (1990)	First detailed investigation of meandering compound channels processes. Need established to model river channel meandering within floodplain belt as a continuum.	
Walling *et al* (1991)		Cs-137 dating technique developed to map high resolution deposition patterns.

Table 2.1: Summary of recent significant advances in floodplain hydraulics and floodplain geomorphology.

2.2 Numerical modelling strategies for compound channel flow

The objective of this discussion is to identify those physically based numerical modelling schemes that have been developed to simulate river channel/floodplain flow at a high degree of spatial and temporal resolution and which could satisfy the basic model criteria stated in Chapter 1. We are therefore concerned with schemes that are distributed in both time and space. Equations for fluid flow problems are based on application of the law of conservation of momentum to a fluid body. This yields one, two or three dimensional systems of partial differential equations that can be solved either analytically, to yield an exact solution, or numerically, to give an approximation to the true solution. In general, the non-linear, non-steady problems for regions of irregular topography encountered when considering flow in open channels are impossible to solve analytically (Huyakorn and Pinder, 1983) and recourse is made to numerical solution methods. Suitable approximation techniques are discussed below (Section 2.2.1). Modelling strategies based on these techniques are then considered (Section 2.2.2) according to the dimensional representation of the flow field under consideration.

2.2.1 Numerical solution techniques

Numerical solution techniques for fluid flow in open compound channels can be divided into three categories:

i) finite difference methods,
ii) finite element methods,
iii) the method of characteristics.

Finite difference methods divide the solution domain into a continuum of adjacent sub-areas termed cells, defined by nodal points at each cell corner. This division is usually based on a regular grid, although more complex schemes allow for deformation of cell boundaries to improve physical representation (Banks and Falconer, 1989). Nodes may be defined by either radial or, more usually, cartesian coordinates where the resulting grid is composed of rectangular cells. Finite difference methods define approximations to a continuous solution at isolated points which are considered representative of the sub-area of which they form part. Consequently it may be necessary to apply additional interpolation schemes to obtain solutions at arbitrary points within a particular sub-area (Pinder and Gray, 1977). The solution is obtained by approximating the variation of differential or partial differential equations to a set of discrete values of the equation derivatives applying over some small interval. This is achieved using an interpolating polynomial or more commonly a Taylor series expansion (Pinder and Gray, 1977), which, with the addition of boundary and/or initial conditions, allows closure of the problem.

Finite difference methods allow the solution of complex differential and partial differential equations in a computationally efficient manner. It has been noted (Zeilke and Urban, 1981), however, that in order to represent complex geometries, for example to resolve a river channel separately from its floodplain, the cell size must be relatively small. This significantly reduces computational efficiency. In addition, to ensure accuracy, solutions require a grid resolution which can represent the steepest gradients of system state variables. As solution resolution cannot be varied within the grid, the system may be over-represented in areas such as low lying floodplains, where these gradients are shallow. Computational efficiency will consequently be less than optimal in these situations.

Finite element methods are also based on subdivision of the solution domain. In this case, however, subdivision takes the form of a flexible grid, usually consisting of triangular or quadrilateral elements. The resulting grid can be either parametric (regular) or isoparametric (irregular). Thus, no restriction is placed on size or shape of elements. This allows complex structures to be represented with a minimum number of elements and solution resolution concentrated in areas of specific interest (see Figure 2.5). Solutions to differential and partial differential equations are then defined over the entire domain by assuming that the equation under consideration varies across each element in a pre-defined manner. These interpolation functions may be linear, quadratic or some higher order polynomial in form and approximately quantize the differential equation on each finite element. Either variational principles or a weighted residual method is then used to transform the governing equations into finite-element equations, or shape functions, governing variation of state variables over each element (Baker, 1983). These local equations are collected into some global matrix within which appropriate boundary conditions are incorporated. This matrix equation system can then be solved to generate nodal values of the state variables. Despite the effectiveness with which finite elements can approximate spatial derivatives, for transient systems this approximation will only hold for one particular time. Consequently, an independent discretization of the time derivative is

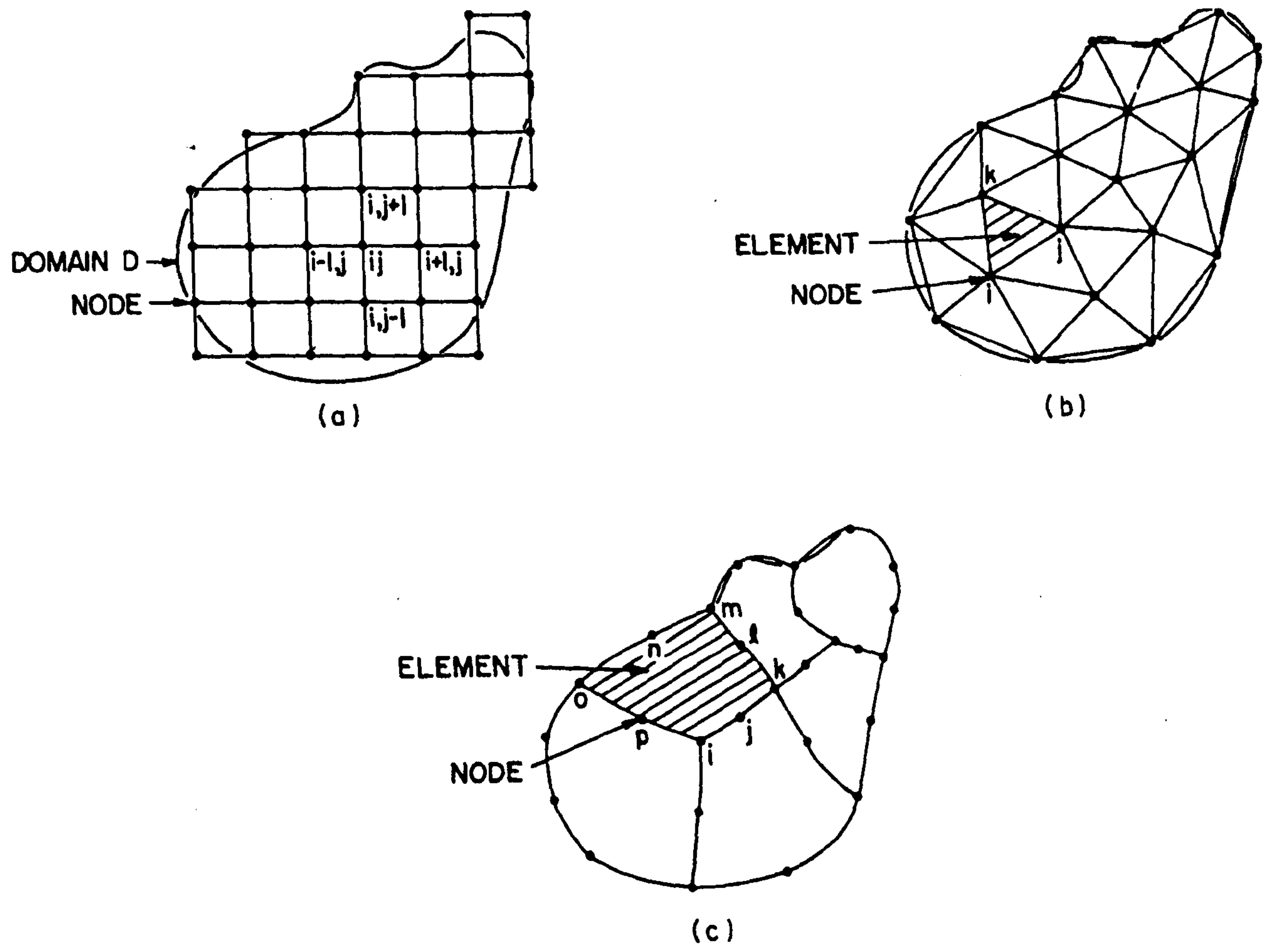


Figure 2.5: Representation of complex topography with finite difference and finite element discretizations. (a) Regular finite difference network. (b) Finite element network using parametric triangular elements. (c) Finite element network using isoparametric quadrilateral elements. From Pinder and Gray (1977).

required to represent non-steady system states. This is most frequently achieved with an appropriate finite difference formulae (Pinder and Gray, 1977), however finite element techniques can also be used. The finite element method therefore possess a number of advantages over finite difference methods (see for example Beven, 1977), however computational demands are higher and their theoretical basis is less well developed.

The method of characteristics has been developed to solve problems concerning the analysis of transient fluid flow where the controlling equations are hyperbolic partial differentials (Huyakorn and Pinder, 1983). The procedure consists of transforming these equation systems into ordinary differentials which may then be solved. A scheme to simulate two dimensional flow with this method has been developed by Schmitz *et al.* (1983) using a rectangular grid capable of variable resolution. However computational requirements were large and it was acknowledged that the necessity for an *a priori* approximation to the streamlines could limit applicability.

Property	Finite element	Finite difference
Irregular element shapes	yes	no
Varying solution resolution	yes	no
Representation of complex topography	*****	**
Solution developed for...	whole area	single point
Relative computational efficiency	**	****
Theoretical basis	**	****

Table 2.2: A comparison of the properties of finite element and finite difference models relevant to open channel flow applications. Each relevant aspect has been rated from * (low) to ***** (high) for each method.

The properties of finite difference and finite element methods are summarised in Table 2.2. **In the consideration of problems involving irregular topography and varying solution resolution finite element methods are shown to have a number of advantages.** River channel/floodplain flow problems have been demonstrated to possess these characteristics. Any modelling strategy identified to deal with such problems should therefore be based on the finite element technique.

2.2.2 Modelling strategies

Section 2.1 demonstrated that flow in open channels is an unsteady, non-uniform problem with a complex three dimensional structure. A complete solution to this problem is provided by the equation of mass continuity and the Navier-Stokes force-momentum equations (see Tritton, 1977, chapter 5). Full solution of these equations is often more computationally and data intensive than is warranted by the problem under consideration. Simpler one and two dimensional forms of the full equations have consequently been developed for application to fluid flow problems where this level of detail is not required. These are considered in Sections 2.2.2.1 and 2.2.2.2 respectively, while full three dimensional solutions are considered in Section 2.2.2.3.

2.2.2.1 One dimensional floodplain modelling

Models and sub-model components of this type solve either the one dimensional St. Venant equations for free surface flows, or some derivative of these. Derivations include simplified hydraulic routing methods, of which the kinematic wave and diffusion wave models (see Cunge *et al.*, 1980; Fread, 1985; Bathurst, 1988) are the best known examples. The most simple one dimensional schemes are based on uniform flow formulae such as the Manning equation. A number of such schemes are now commercially available and these form the only currently functional analytic tool for river flood studies at the long reach scale (Samuels, 1990). These are all built from similar components; the St. Venant equations, a numerical discretization scheme and a set of site specific computational algorithms.

The St. Venant equations (De Saint Venant, 1871) estimate flow for specific cross-sectional locations. Lateral flows are not accounted for as the model assumes that the flow path and distribution can be determined *a priori* (Gee *et al.*, 1990). This assumption means that lateral momentum transfer between main channel and floodplain flows, or the complexities of overbank flow in meandering compound channels, are not accounted for. Site specific computational algorithms do, however, allow free surface wind stress and lateral inflow effects to be included (Skeels and Samuels, 1989).

Solution schemes for the St. Venant equations are usually based on finite difference methods such as the Abbott-Ionescu (Abbott and Ionescu, 1967), Vasilev (Vasilev and Gudonov, 1963) or Preissmann (Preissmann, 1961) schemes. More recently Goussebaile and Lepeintre (1989) have developed a solution based on the method of characteristics, however this solution has only been developed for rectangular channels and extension to natural geometries may be problematic. Numerical schemes have therefore concentrated on finite difference procedures. The stability and error inherent in these methods has been examined by Skeels and Samuels (1989). They showed the Preissmann scheme to be most advantageous, particularly where flow may become supercritical.

Model solutions for the St. Venant equations are typically developed for a series of discrete cross sections, with the success or otherwise of the model depending to a large extent on the siting of these (Samuels, 1990). Cross section spacing should be sufficiently frequent to describe the hydraulic behaviour of the reach in question. This can be disadvantageous, firstly, because it is a subjective process and secondly, because areas between cross sections will not be fully described. It is therefore difficult to describe processes, such as inundation, that show a high degree of spatial variation. Samuels (1990) has produced rules for cross section location in one dimensional models by examining the interaction of key terms in the St. Venant equations. This has allowed some of the problems discussed above to be addressed. However this study was confined to an examination of channel-only models and it was acknowledged that topological discretization of the floodplain and its linkage to channel flow could lead to further problems for one dimensional schemes.

One dimensional models differ in their treatment of floodplain areas (see Figure 2.6). These have been treated in three quite distinct ways; as part of a single channel unit, as a storage area or as a discrete routing zone:

i) Single channel unit - this approach treats channel/floodplain flow in an identical fashion to channel only flow, despite its distinctly different hydraulic

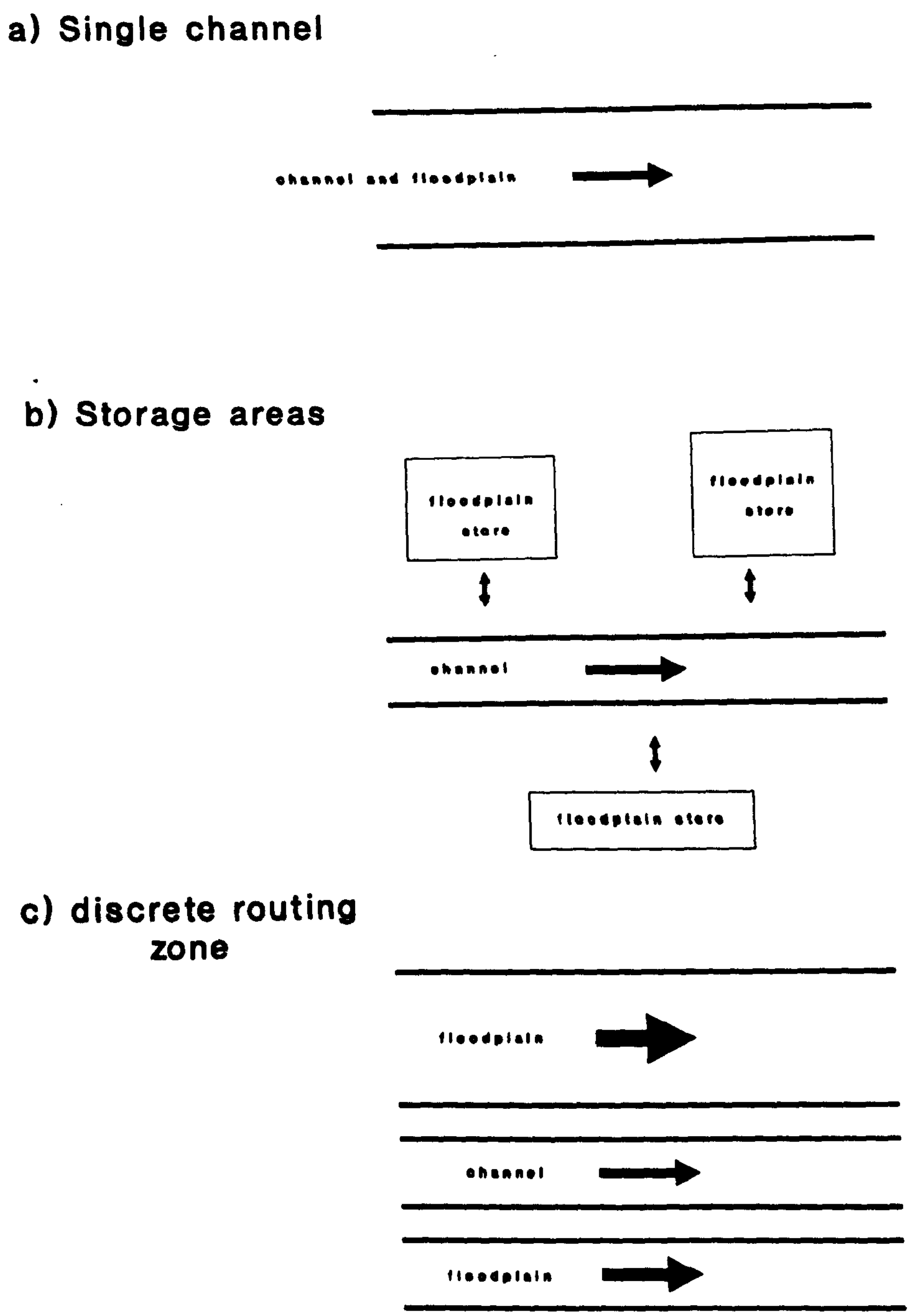


Figure 2.6: Representation of floodplain areas in one dimensional flow models. Currently these are the only available schemes for the simulation of river channel/floodplain flow at the long reach scale.

behaviour. In the SHE model (Abbott *et al.*, 1986) for example, flow is allowed to extend laterally to a pre-set limit, approximately half the width of the floodplain. This area is then treated as a single stage channel with complex geometry.

ii) Storage area - models such as HEC-1 (HEC, 1981), FLUCOMP (Samuels, 1983a), FLOODTIDE (Neat *et al.*, 1989) and the model proposed by Pender and Ellis (1990) treat any water that spills out of the defined channel as going into storage. In the HEC-1 model this stored water is allowed to return to the channel as infiltrated baseflow, however in the FLUCOMP model the amount of storage is represented by a coefficient which is then used to adjust the routing procedure. This can be an acceptable solution in some physical systems such as washlands or embanked rivers (Pender and Ellis, 1990), however in most river channel/floodplain systems the floodplain system is not a series of discrete storage areas but interacts dynamically with main channel (see Section 2.1).

iii) Discrete routing zone - HYMO3 (Williams and Hann, 1973; Baird and Anderson, 1990), EMBER (Samuels, 1983a) and the model proposed by Fread (1976) consider floodplain flow as being separate from the main channel. This can then be routed downstream under a separate set of controlling conditions. This allows floodplains to convey flow and accounts for effects such as different boundary friction resistances on the floodplain and the more direct flow path of the floodplain relative to the main channel. It does not, however, allow momentum exchange effects to be accounted for. An attempt has been made to account for momentum exchange effects in the HYMO3 model (Baird and Anderson, 1990), based on shear stresses acting across an imaginary interface between the floodplain and channel units. This is, however, an analytical solution derived from straight channel flume studies (in particular Knight and Hamed, 1984) and may not necessarily be representative of natural channels.

One dimensional approaches to river channel/floodplain flow therefore provide a simple and computationally stable means of gaining information on water level changes throughout flood events. When considering unsteady flow routing in river channels where lateral flows can be assumed to be negligible, the approach performs adequately. It has been demonstrated, however, that in such modelling schemes:

i) a number of flow phenomena are not taken into account by the controlling equations; these include secondary currents, momentum exchange effects and the set of processes associated with meandering compound channels (Section 2.1.2.2),

ii) the representation of floodplain areas is frequently unrealistic,

iii) schematization of topographic features is often poor (Bathurst, 1988),

iv) grid resolution may be too low to enable processes of geomorphological and hydraulic interest, such as inundation, to be effectively simulated.

2.2.2.2 Two dimensional floodplain modelling

Two dimensional approaches to floodplain modelling have been investigated as a solution to some of the problems inherent with one dimensional schemes. Although numerical and computational demands are increased, such approaches are capable of simulating a large set of complex flow problems which one dimensional schemes could not effectively undertake. Such solutions are based on the Navier-Stokes equations reduced to two dimensional forms, usually in the horizontal plane, to simulate laterally distributed flow parameters.

Depth-averaged Navier-Stokes equations are able to solve both steady and unsteady river flow problems (Vreugdenhill and Wijbenga, 1982) and can be modified to account for such effects as turbulence, wind stress and boundary friction. Solution of the resulting equations provides a prediction of both the flow depth and the two dimensional velocity vector at each computational point in the solution grid. In their derived form the Navier-Stokes equations only apply to laminar flow but are adapted to account for turbulent flow by following some statistical averaging procedure and by making some approximation to the Reynolds stresses. Two dimensional models generally utilize a first order approximation to the Reynolds stresses (Samuels, 1989). This simple turbulence model is derived from the Boussinesq approximation, where the additional force acting on the viscous term in the Navier-Stokes equation during turbulent flow is estimated through an equivalent eddy viscosity coefficient. Higher order Reynolds stress approximations, such as the depth averaged K-ε turbulence model (Keller and Rodi, 1988), have been considered. However, solution of this type of turbulence model requires a large number of calibrated empirical coefficients and the approach has, therefore, generally been restricted to three dimensional modelling (see Section 2.2.2.3). The final numerical model configuration can take a number of forms depending on the assumptions used in derivation of the dynamic equations (Samuels, 1989). In an analytic study of two of these forms Samuels (1989) has shown that traditional two dimensional formulations artificially enhance the strength of eddy zones. Advantages were shown with an alternative equation based on the unit flow vector and water depth. However, this approach has not been sufficiently developed to justify inclusion in model codes. Hervout (1989) has compared the two dimensional, finite element TELEMAC code to experimental flume results. This study showed that the discrepancy between observed and predicted results was always less than ±8%, with measurement errors less than ±1%, indicating the high accuracy that can be achieved with two dimensional models.

Solution schemes for these equations have been based on both finite difference (see Huyakorn and Pinder, 1983; Peyret and Taylor, 1983) and finite element methods (see Pinder and Gray, 1977; Baker, 1983; Pironneau, 1989), the relative merits of which have been discussed in Section 2.2.1. Finite difference approximation schemes have until recently dominated two dimensional flow problems (Peyret and Taylor, 1983), however research has largely been confined to estuaries (e.g. Zielke and Urban, 1981; Chatterjee and Chakraborty, 1989) and harbour environments (e.g. Hervout, 1988; Fritsch *et al.*, 1989; Nece and Falconer, 1989). In such situations there is little need to model an irregular shoreline or include complex topographic features in the spatial discretization. In addition, the need to vary solution resolution over the grid is reduced, due to low gradients of system state variables. Finite difference techniques are therefore an adequate approximation to these particular physical systems. When considering river channel problems, however, such factors have to be taken into consideration and the application potential of finite difference schemes is consequently limited. To overcome this, finite element techniques have developed rapidly since initial applications to fluid flow problems by Oden (1972), Olson (1972) and Baker (1973). Finite element methods have, however, only been applied to certain classes of river flow problems. These have included analysis of detailed flow patterns near engineering structures (Tseng, 1975; King and Norton, 1978), river confluence studies (Niemeyer, 1979; Su *et al.*, 1980), estuary studies (Johnson and Thompson, 1978; Holtz and Nitsche, 1980) and river channel/floodplain modelling (Samuels, 1985; Akanbi and Katopodes, 1988). In all these studies the scale of interest has been confined to reaches a few river widths in length and rarely has the river channel been resolved separately from the floodplain (Gee *et al.*, 1990). Some progress in this direction has been reported by Gee *et al.* (1990) who have developed a prototype schematization of a two dimensional finite element model for a 24 km reach of the River Fulda, Federal Republic of Germany. This demonstrated the potential of the finite element approach to overcome these deficiencies. We may conclude that relatively little research into two dimensional approaches to river channel/floodplain flow problems has been directed towards the research needs identified in Chapter 1, despite the obvious benefits which would result.

Two dimensional models represent compound channels as a single continuous region. This resolves a number of the problems associated with one dimensional approaches. For example, two dimensional approaches are able to model the momentum exchange mechanism between main channel and floodplain flows. In addition, significant two dimensional effects occurring in meandering compound channels, such as channel flow spilling onto floodplains before re-entering the channel, can be simulated. Two dimensional models cannot, however, simulate the secondary currents, downstream effects or flow expansion and contraction present in meandering compound channel flow. Similarly, the description of momentum transfer is a two dimensional approximation to a three dimensional problem.

The approach therefore has a number of distinct advantages when considering the modelling of river channel/floodplain reaches. These are:

i) Significant flow phenomena not included by one dimensional approaches, such as momentum exchange effects, can be simulated by such models.

ii) Two dimensional velocity vectors and water surface elevations can be calculated for each computational node. This has obvious potential for application to geomorphological investigations.

iii) Floodplain areas are treated in a realistic fashion.

iv) Two dimensional finite element models can incorporate complex small scale topographic features, such as bridge piers, into the spatial discretization.

v) The approach has been evaluated for a large number of field and experimental situations.

Despite these advantages, two dimensional approaches are considered inadequate in a number of respects:

i) They only provide a partial description of a three dimensional flow field.

ii) The representation of Reynolds stresses is only to a first order approximation.

iii) Such models have only been applied at a small scale, thus issues relating to the inclusion of complex topographic detail at the long reach scale have not been adequately resolved.

iv) Few two dimensional models of river channel problems have resolved the channel separately from the floodplain, despite this being an essential step in the simulation of flow in meandering compounds channels.

2.2.2.3 Three dimensional floodplain modelling

Three dimensional approaches to compound channel flow have been constructed for problems where a near complete description of the flow field is essential. This process allows a number of the limitations associated with two dimensional approaches to be overcome. In particular, phenomena such as secondary circulation and formation of shear layers in compound channels can be realistically simulated. However, at the present time three dimensional flow models are, almost exclusively, research tools at an early stage of development and this leads to a number of problems with their application.

Three dimensional floodplain modelling is based on the assumption of parabolic flow, whereby a predominant flow direction is presumed to exist. This assumption allows the parent three dimensional, time averaged, Navier-Stokes equations to be simplified into parabolic form (Baker, 1983). This system of equations is then typically closed using an algebraic stress or K-ε turbulence model (Launder and Ying, 1973; Naot and Rodi, 1982; Demuren and Rodi, 1984) to approximate turbulent Reynolds stresses. Prinos (1990) has compared a number of algebraic stress models and found the

Demuren-Rodi model to be the most generally applicable. However, in general, Shiono and Knight (1991) state that solution of such models requires a large number of empirical coefficients and this limits their application to topographically complex field studies.

Solution schemes for three dimensional flow equations are based on two methods; finite elements (see Baker, 1983, chapter 7) and finite volumes (see Patankar and Spalding, 1972). Of these approaches finite volume methods have received most attention in relation to compound channel problems (see for example Prinos, 1989; Prinos, 1990). The momentum equations are solved with an efficient marching forward solution. Calculations start at an upstream cross section with known initial conditions and proceed downstream until fully developed flow occurs. Alternatively, a solution can be developed for single compound cross sections (Prinos, 1990). Here uniform distribution of all variables is prescribed and step-by-step integration performed until fully developed flow is generated. Solutions for three dimensional parabolic Navier-Stokes equations have, however, only been developed for steady flow situations.

Three dimensional studies of compound channel flow have generally concentrated on simulation of experimental cross sections (Prinos, 1989; Prinos, 1990) in order to investigate aspects of the momentum transfer mechanism in open, compound channels (see for example Lau and Krishnappan, 1986; Larson, 1988; Kawahara and Tamai, 1989; Tominaga *et al.*, 1989). Predictions from these studies have shown satisfactory agreement with experimental measurements and the models seem able to simulate the complex secondary currents occurring in compound channels. These studies have not, however, considered channel reaches or situations of complex topography.

Three dimensional modelling therefore has the advantage that it is able to provide a near complete description of the flow field in compound channels and allows a more realistic treatment of turbulent flows by means of a higher order approximation to the turbulent Reynolds stresses. The approach is, however, still under development and suffers from a number of limitations:

i) algebraic stress models require a large number of calibrated empirical coefficients,

ii) solutions are computationally intensive,

iii) the approach has only been applied to cross sections or very short reaches,

iv) full dynamic simulations cannot be performed.

2.3 Identification of a modelling strategy

Section 2.1 of this Chapter has identified two significant processes which any model of river channel/floodplain flow must include to meet the design criteria outlined in Chapter 1. These are the momentum exchange mechanism between main channel and floodplain flows identified by Sellin (1964) and the delivery of water onto the floodplain and its subsequent return to the channel.

Section 2.2 has identified modelling approaches currently used to simulate river channel/floodplain flow. The characteristics of these approaches have been examined and the problem classes to which they apply have been delimited. These properties are summarised in Table 2.3. A modelling strategy for the high resolution simulation of river channel/floodplain flow over long reach lengths, that satisfies the model design criteria outlined in Chapter 1, may now be identified.

One dimensional open channel flow models are unable to fulfil our basic criteria for an inundation modelling scheme. The controlling St. Venant equations are unable to provide a sufficiently detailed description of the problem hydraulics, making no account of lateral variations in the flow field. Thus, one dimensional approaches cannot predict flow velocities or inundation patterns and make no account of either process identified above. In addition, the finite difference discretization schemes typically used in such models are unable to represent the complex topography characteristic of river channel/floodplain flow problems.

Two dimensional approaches are able to account for momentum transfer in compound channels and the two dimensional flow effects associated with river meanders. Furthermore, the controlling equations can simulate velocity fields and inundated areas at a high level of spatial resolution. Numerical schemes to solve the depth averaged Navier-Stokes equations have been based on both finite difference and finite element approaches. However, this Chapter has demonstrated that for river

Property	1-d models	2-d models	3-d models
Simulation of unsteady flow	yes	yes	no
Representation of complex topography	**	*****	*
Incorporation of momentum transfer	no	yes	yes
Resolution of channel seperate from floodplain as part of an interacting flow region	no	no	no
Potential ability to simulate meandering compound channel effects	no	yes	no
Grid resolution	*	****	*****
Computational demands	**	****	*****
Ease of application	yes	yes	no
Application to long reach lengths	yes	no	no
Application to natural systems	yes	yes	no

Table 2.3: A comparison of one, two and three dimensional approaches to the modelling of river channel/floodplain flow problems. The utility of these approaches have been scored from * (low) to ***** (high) in respect of each relevant category.

channel/floodplain flow problems finite element techniques have a number of distinct advantages. Two dimensional finite element approaches therefore have a unique potential to effectively undertake the representation of complex topography necessary for simulation of river channel/floodplain flow and, additionally, are able to represent floodplain areas in a realistic, continuous, fashion (Gee *et al.*, 1990). In general, however, such schemes have only been applied to short river reaches and have not resolved the river channel separately from the floodplain. Thus, two dimensional models have not been used to model the delivery of water to the floodplain and its subsequent return to the channel.

Three dimensional approaches provide a near complete description of the flow field, however such schemes are unsuitable for application to river channel/floodplain flow problems for a number of reasons. Firstly, three dimensional models are only capable of steady state simulations. Consequently they cannot describe dynamic changes in velocity fields and inundation zones occurring throughout flood events. Chapter 1 has identified this as a major research requirement of this project. Secondly, such schemes have only been applied to channel cross sections and not to river reaches. Lastly, the approximations used to represent Reynolds stresses in three dimensional models utilize a large number of empirical coefficients that require extensive calibration. Thus, three dimensional schemes, at their present stage of development, are not robust enough to deal with the complex topography and dynamic flow environment of natural river systems. The additional modelling effort necessitated by such an approach is therefore unjustified.

We may conclude that two dimensional, finite element approaches are the most appropriate means of simulating flow processes in meandering compound channels. Thus far, however, such schemes have not been developed to model high resolution river channel/floodplain flow over long reach lengths, despite their obvious potential for this task. To effectively undertake this type of modelling, enhancements to a prototype two dimensional finite element scheme are required that:

i) extend this modelling strategy to consider river reaches of 10 - 30 km,

ii) develop finite element discretizations capable of resolving river channels meandering within a wider floodplain belt,

iii) develop finite element discretizations capable of incorporating complex topographic features at the long reach scale.

No currently available model of open channel flow has this capacity and as a consequence the research needs identified in Chapter 1 have not, to date, been addressed. **The originality of this project therefore lies in the identification and further development of a two dimensional finite element scheme uniquely capable of meeting these needs. This will be undertaken in Chapter 3.**

CHAPTER 3

Model selection and development

Two dimensional finite element flow models were identified in Chapter 2 as the most appropriate high resolution simulation method for flood inundation over long reach lengths. This Chapter reports on the selection and further development of an appropriate prototype scheme. Chapter 2 has demonstrated the need for developments that:

i) extend two dimensional finite element strategies to consider river reaches of 10 -30 km,

ii) develop finite element discretizations capable of resolving river channels meandering within a wider floodplain belt,

iii) develop finite element discretizations capable of incorporating complex topographic features at the long reach scale.

Having identified a suitable prototype model and implemented the above developments, a research design for the configuration, evaluation and application of this enhanced scheme will be proposed.

3.1 Model Selection

Chapter 2 has demonstrated that an appropriate prototype two dimensional finite element model for river channel/floodplain problems should possess the following specific characteristics:

i) The scheme should be able to simulate steady and unsteady flow.

ii) The scheme should be able to account for varying boundary friction and turbulence characteristics over the mesh.

iii) The scheme should incorporate momentum exchange effects.

iv) The scheme should simulate the delivery of water onto the floodplain and its subsequent return to the channel. In effect this means that the selected model should be able to simulate a horizontally moving flow boundary, allow areas of the mesh to be either wet or dry and simulate a smooth transition between these two states.

In addition, it would clearly be of benefit if the prototype scheme were to be:

v) thoroughly evaluated for small reach lengths and topographically simple problems,

vi) able to account for a variety of numerical model boundary conditions.

These criteria preclude model construction from first principles as coding and evaluation of such a model could not be accomplished within the timeframe of this project and would merely serve to duplicate existing research. Model codes for numerical solution of the two dimensional flow equations already exist and have been evaluated for the restricted set of problems outlined in Section 2.2.2.2. A commercially available prototype code that includes the above criteria and could potentially be extended to the simulation of flood inundation problems at the long reach scale was therefore selected as the basis for this project.

Few available models meet the above criteria. The majority reported in the literature (see for example Tseng 1975; Niemeyer, 1979; Akanbi and Katopodes, 1988) are either problem specific or still at a developmental stage and therefore not sufficiently evaluated. Two suitable models were identified (Baird and Anderson, 1990); the FLOUT model developed at Hydraulics Research Ltd., Wallingford (Samuels, 1983b) and the RMA-2 package (King and Norton, 1978), developed for the US Army Corps of Engineers. These more generally applicable codes, both available commercially, were therefore evaluated to determine their potential for development. Both schemes were conceptually similar, however FLOUT was unable to simulate a horizontally moving flow boundary. This is essential as floodplain areas are characterized by small longitudinal and lateral slopes which cause large variations in flow boundary position during passage of a flood wave. Recourse can be made to deformable boundary techniques to overcome this problem (see for example Lynch and Gray, 1980), however these cause slow solution convergence and necessitate additional grid detail in floodplain areas (King and Roig, 1988). Consequently, model solutions such as FLOUT have removed elements from the solution when depth at a single node in any element became equal to, or less than, zero. This has the benefit of computational simplicity but generates large discontinuities in cross sectional area and predicted shoreline. In order to avoid this, RMA-2 retains elements within the solution as they dewater (see Section 3.2.1.2). This is a significantly better approximation to the physical system than that provided by alternative schemes. In addition, elimination of numerical irregularities within the solution will potentially improve numerical stability for floodplain applications. The RMA-2 model was therefore selected on the basis of this ability.

3.2 Outline of the RMA-2 modelling scheme

RMA-2 is unique in its potential for application to river channel/floodplain flow problems. This section outlines the characteristics of the chosen scheme, which consists of two parts; a physically based model comprising the governing equations and associated parameters and an appropriate numerical solution scheme.

3.2.1 Outline of the physical model

3.2.1.1 Governing equations

RMA-2 solves the Reynolds form of the Navier-Stokes force-momentum equations reduced to two dimensions, in conjunction with the two dimensional mass continuity equation. The Navier-Stokes equations for laminar flow result from application of Newton's second law to an elemental fluid mass. This sets the inertial force of a fluid body equal to the sum of the applied forces. The latter consist of pressure forces, gravitational forces and the frictional force due to fluid viscosity. For a fluid of constant density this results in a system of second order partial differential equations applying to three dimensions, given in eularian form (ie. where the observer's frame of reference is temporally stationary) by cartesian vector notation as:

Force-momentum equation:

$$\rho \frac{D\mathbf{u}}{Dt} = -\nabla p + \mu\nabla^2\mathbf{u} + \mathbf{F} \tag{3.1}$$

Continuity equation:

$$\nabla \cdot \mathbf{u} = 0 \tag{3.2}$$

Where: ρ = fluid density; $\mathbf{u}$ = velocity; t = time; p = pressure; μ = viscosity; $\mathbf{F}$ = the set of terms (for example gravity) to be included in the specification of particular problems.

A number of modifications are then required to apply this equation system to a turbulently flowing fluid mass existing on a rotating sphere in the presence of external tractive forces.

It has been demonstrated (Section 2.1) that velocity and pressure in a turbulent flow field vary gradually or rapidly in both time and space. For bulk flow problems, where we are not concerned with simulating this micro-scale structure, we can replace dependent variables in the Navier-Stokes equations with their mean value evaluated over some time interval. An extra term is also added to the viscous force component of the Navier-Stokes equations due to the increased internal shear, or Reynolds, stress of a turbulent fluid (see equation 2.4). These modifications result in the Reynolds form of the Navier-Stokes equations. Solution of this system of equations involves determination of these stresses. In practice, however, this requires measurement of the three dimensional velocity field of the flow (Shiono and Knight, 1991), which is effectively impossible. The Boussinesq approximation (see Massey, 1983, p150) is therefore introduced to make the Reynolds equations mathematically tractable. The Reynolds stress in a particular plane is equated to a turbulent exchange coefficient, ε, dimensionally similar to the coefficient of viscosity, μ, and therefore termed the eddy viscosity coefficient. This may then be estimated from physical criteria.

The Reynolds equations are derived for an elemental fluid mass. In order to allow application to natural flow situations, terms must be introduced that reflect the additional constraints of physical systems. These comprise external traction terms representing coriolis force, wind stress and boundary friction. This three dimensional system of equations may then be vertically integrated to yield depth averaged two dimensional forms. These resulting equations assume (Tritton, 1977; Samuels, 1983b; French, 1986):

i) the fluid is incompressible and of constant density,

ii) vertical velocity and acceleration are negligible,

iii) the river bed does not change with time.

The final form of the governing equations in cartesian coordinates has been outlined by King and Norton (1978) as:

Force-momentum equations:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial h}{\partial x} + g\frac{\partial z_o}{\partial x} - \frac{\varepsilon_{xx}}{\rho}\frac{\partial^2 u}{\partial x^2}$$

$$- \frac{\varepsilon_{xy}}{\rho}\frac{\partial^2 u}{\partial y^2} + \left[-2\omega v \sin\lambda + \frac{gu}{C^2 h}(u^2+v^2)^{1/2} - \frac{\zeta}{h} V_a^2 \sin\psi \right]$$

$$= 0 \qquad (3.3)$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial h}{\partial x} + g\frac{\partial z_o}{\partial x} - \frac{\varepsilon_{yx}}{\rho}\frac{\partial^2 v}{\partial x^2}$$

$$- \frac{\varepsilon_{yy}}{\rho} \frac{\partial^2 v}{\partial y^2} + \left[2\omega v \sin\lambda + \frac{gv}{C^2 h} (u^2+v^2)^{1/2} - \frac{\zeta}{h} V_a^2 \sin\psi \right]$$

$$= 0 \qquad (3.4)$$

Where the terms within [ ] are the external traction terms (coriolis effect, bed friction and surface wind stress.

Continuity equation:

$$\frac{\partial h}{\partial t} + \frac{\partial}{\partial x}(uh) + \frac{\partial}{\partial y}(vh) = 0 \qquad (3.5)$$

Where: ρ = fluid density; u, v = velocity components in the x and y directions; h = water depth; C = Chezy coefficient; g = gravitational constant; z_o = bottom elevation; ε_{xx}, e_{xy}, ε_{yx} and ε_{yy} = turbulent eddy viscosity coefficients; x, y = cartesian coordinates; t = time; ω = rate of earth's angular rotation; λ = local latitude; ζ = empirical coefficient; V_a, ψ = local wind velocity and direction.

Note that in the interests of clarity the bar (average) notation has been dropped. It should therefore be understood that the values of the independent variables represent their **mean condition.**

This particular model formulation does not conserve mass at all points, however King and Norton (1978) state that mass continuity along outer boundaries of the finite element mesh is maintained.

3.2.1.2 Wetting and drying of the finite element mesh

RMA-2 has a unique approach to wetting and drying of the finite element mesh that renders it particularly suitable for application to floodplain environments. For such areas King *et al.* (1986) have proposed an element elimination procedure that approximates the inundation process in a significantly more realistic fashion than conventional methods. King and Roig (1988) outline the incorporation of this procedure into the physical model. The governing equations are modified for elements that are alternately flooded and exposed during a simulation. For elements in transition between wet and dry states a domain coefficient, σ, is defined to scale simulated water volume to the true volume of water residing on the partially wet element during each time step. This coefficient is conceptually similar to the porosity term in Darcy flow through porous media, reflecting the proportion of the flow domain available for fluid movement. σ is then allowed to fall to zero as an element dewaters. Two numerical difficulties arise with this approach. Firstly, solution stability is reduced as nodes with zero depth are incorporated into the finite element continuum. Secondly, σ is discontinuous when water surface elevation, h, is equal to bottom elevation, z_0. Consequently, the derivative $\partial\sigma/\partial h$ is undefined at this point, introducing additional numerical difficulties. These problems are solved by making the following approximations. A small positive water depth is maintained at each node while the element of which it is part remains within the solution. This is achieved by defining some minimum value of σ, σ_{min}, which is small and positive and represents nearly dry conditions. A discontinuity at $h = z_0$ is avoided by allowing σ to vary from the fully wet condition, $\sigma = 1$, to the nearly dry condition, $\sigma = \sigma_{min}$, over some small range of h termed R. In physical terms R describes the subgrid bathymetry in the vicinity of a particular node that causes the parent element to be partially wet or dry. King and Roig (1988) therefore state that the additional parameters, σ_{min} and R, can be estimated from physical data. A summary of these modifications is given in Figure 3.1. Initial testing of these new parameters has been carried out by Gee *et al.* (1990). A single case study was used to demonstrate enhanced model stability for floodplain applications. Information on sensitivity of the new parameters has not, however, been collected despite the possibility of using R to calibrate the model for variations in microtopography at a subgrid scale. **It would therefore be desirable to undertake additional field validation of the modified wetting and drying routine in RMA-2 and rigourously test its operation.**

3.2.1.3 Parameterization of the physical model

From the controlling equations it can be seen that model simulations require determination of the following physically based parameters:

i) boundary friction coefficient,

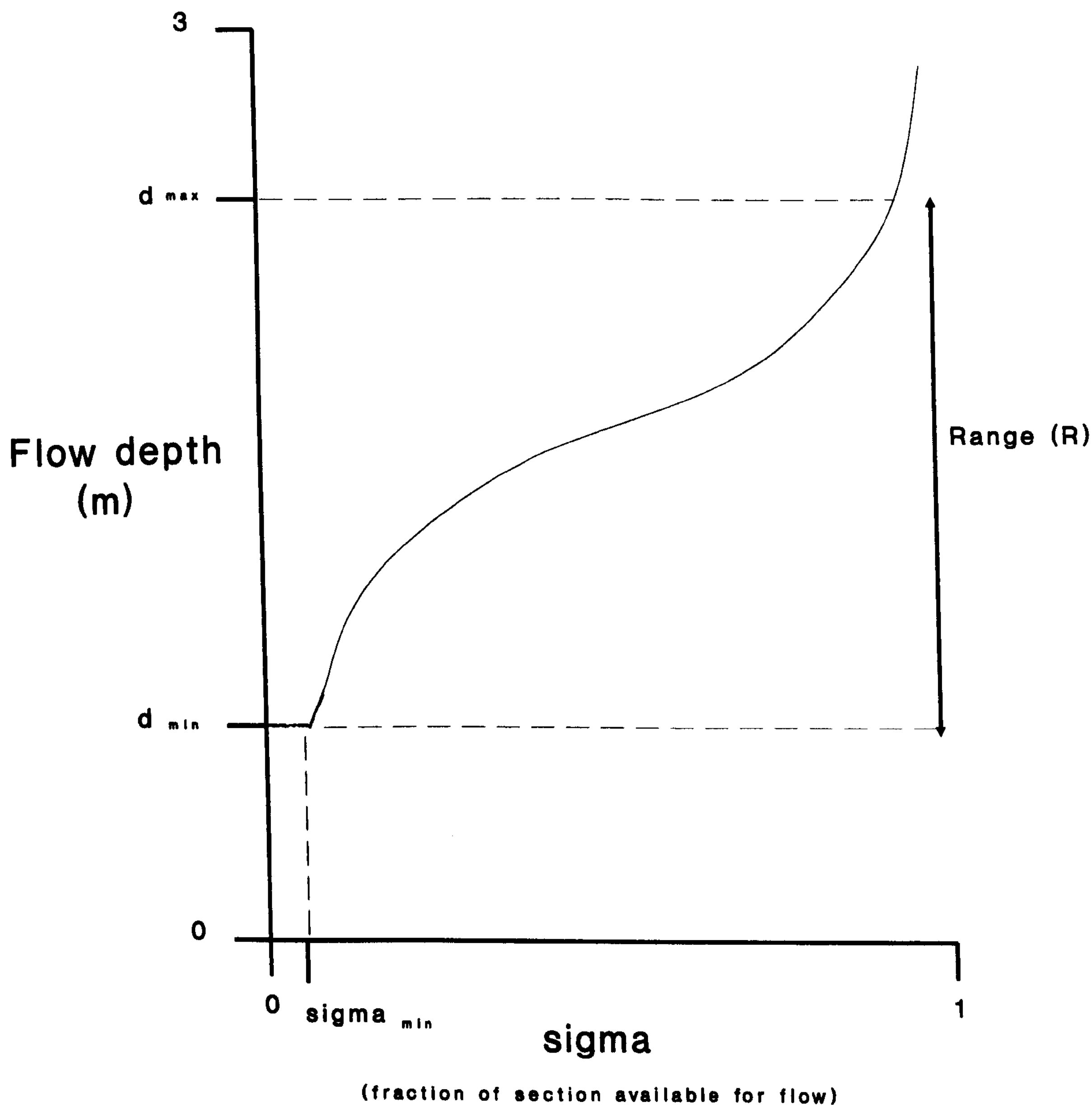


Figure 3.1: Modified wetting and drying routine for RMA-2 indicating the behaviour of the parameters σ, the element domain coefficient, and R, a depth range, as an element dewaters. σ is allowed to fall from 1, representing fully wet conditions, to σ_{min}, representing nearly dry conditions over a small depth range, R (d_{max}-d_{min} on the diagram). As σ does not fall to zero a small positive water depth, d_{min}, is maintained over transition elements. This ensures solution stability for moving boundary problems.

ii) wind stress coefficients (various formulations),

iii) local latitude,

iv) turbulent eddy viscosity coefficient,

v) minimum domain coefficient,

vi) domain coefficient depth range.

Of these all but the surface wind stress coefficients have been included in the model simulations developed in this thesis. The remaining parameters have been evaluated on the basis of physically derived data. Table 3.1 presents a summary of the determination method, typical range of field values, measurement error and the number of values of each parameter required by the model. This demonstrates that turbulent eddy viscosity and boundary friction can be varied spatially across the mesh by assigning each element to one of ten classes with specific friction and turbulence characteristics. In addition, turbulent eddy viscosity values can either be input as an absolute magnitude or internally calculated from the square of principal axis length for each element multiplied by an empirical scaling factor set by the user.

3.2.1.4 Boundary conditions

In order to close the controlling equations, boundary conditions need to be specified that define all inputs and outputs to the physical system. Inputs to the RMA-2 model are defined as an input per time step, however outputs can be defined variously as a discharge per time step, a stage per time step or as a stage-discharge rating curve. These are applied at the end of a control structure created by the modeller to ensure that the flowlines of water entering or leaving the mesh are parallel with the mesh sides and are of uniform magnitude across the section. For the upstream input this is unrealistic as ideally a velocity peak should occur in the channel. Model simulated flows therefore do not become fully developed until a short distance downstream. For the upstream input the control structure allows the point where the boundary condition is applied to be moved downstream of the point where model predictions are extracted for comparison to field data. The boundary conditions are used in the model to define the physical domain within which the governing equations are solved by specifying one or other of the unknowns present in the controlling equations, in this case stage or discharge. This allows the flow component in the momentum equations to be eliminated at this point and moved to the right hand side of the equation. Boundary condition operation is therefore straightforward for stage or discharge conditions specified at each time step, however in the case of a stage-discharge rating curve applied at the downstream end of the mesh the situation is more complex. Here a non-time dependent relationship between discharge and stage is used to generate time dependent boundary condition values of these unknowns from estimates of the flow at the next time step for each node derived using the modified Crank-Nicholson implicit finite difference procedure outlined in Section 3.2.2

Parameter	Number of values required	Determination method	Typical measurement error	Typical percentage error
Boundary friction (Manning's *n*)	10	Photographic definition method (Chow, 1959)	±0.005	±3.3%
Turbulent eddy viscosity	10	Empirically derived or internally calculated	-	±20%
Local lattitude	1	Topographic maps	±0.1° of arc.	±0.03%
Minimum domain coefficient (θ_{min})	1	Empirically derived	-	±5%
Domain coefficient depth range	1	Topographic survey	±150 mm	±10%
Rating curve	see text	Single/divided channel HYMO3 Rated section HR design manual	- - - -	±25% ±15% ±5% ±3%
Stage hydrograph	1 per time step	Gauging station data	±3 mm	±1
Discharge hydrograph	1 per time step	Gauging station data and rating curve	-	±4% - ±26%

Table 3.1: Parameter determination method and typical associated measurement errors for RMA-2 applications.

Determination methods and associated error for these data are also given in Table 3.1.

3.2.2 Outline of the numerical solution scheme

Numerical solution of the governing equations is based on a grid consisting of triangular and quadrilateral isoparameteric elements, with six or eight nodes respectively. Element sides are therefore approximated to a parabola or are linearly interpolated between vertices if mid-side nodes are not specified. The physical system is described in terms of the velocity components, u and v, and the water depth h at corner and mid-side nodes. The model then assumes a quadratic interpolation function for all dependent variables. These interpolation functions approximately quantize the governing partial differential equations over each element. The Galerkin method of weighted residuals (see Pinder and Gray, 1977, chapter 3) is then used to restate each governing equation in terms of shape function weighting factors that satisfy both the boundary conditions and the interpolation functions. The Galerkin method differs from other weighted residual approaches in that the interpolation functions are themselves chosen as the weighting functions. The discrepancy between the true solution and that approximated by the weighting function is defined as a residual, Γ. This residual will tend to zero as the weighting function approaches the true solution. The final procedure of the Galerkin method consists of forcing this residual to zero in an average sense. This results in development of a single equation for each node representing the sum of contributions from all adjacent elements. These local equations are then collected into a global matrix, or Jacobian, incorporating appropriate boundary conditions, which can then be solved simultaneously by numerical integration.

An iterative procedure is required for solution of the Reynold's equations due to their extreme non-linearity, with estimates of u, v, and h being updated after each time step until convergence is achieved. RMA-2 adopts the Newton-Raphson iterative procedure as this has proved to be convergent for all problems that show reasonable answers in the initial step (Norton *et al.*, 1973). Alternative methods based on successive approximations were examined by this study and shown to be unstable for some test cases. The Newton-Raphson method (see Huyakorn and Pinder, 1983, chapter 4) produces new estimates of the problem under consideration on the basis of the error of the last solution. This is defined as an error function, f. The Jacobian of the system at a particular iteration is used to indicate the gradient of f with respect to changes in the dependent variables. This gives the rate of change in the error of a solution at a particular iteration. New estimates are produced by calculating corrections to this error gradient that force the function to zero. Figure 3.2 gives a schematic representation of this procedure for a one degree of freedom system. In real systems with m degrees of freedom there are m slopes of f. The solution therefore adjusts all values of f simultaneously in order to calculate the new estimate.

Section 2.2.1 has demonstrated that in order to represent non-steady system states the spatially discretized finite element method requires an independent discretization of the time derivative. RMA-2 achieves this with a simple implicit finite difference procedure, specifically a modified Crank-Nicholson method (see Norton *et al.*, 1973). This procedure assumes that the velocity vector, **u**, is known at time t. For a solution at time $t + \Delta t$, acceleration is assumed to vary linearly over a time interval ϕ, where $\phi = \theta\Delta t$ and $\theta \geq 1$. This allows a differential equation for **u** at time $t + \Delta t$ to be written for each computational node. These may then be collected and solved simultaneously. Norton *et al.* (1973) state that this solution is unconditionally stable where $\theta > 1.37$ however, accuracy will depend on how well the assumption of linear acceleration represents the physical system. For problems involving gradually varied, unsteady flow, for example the passage of a flood wave, this would appear to be a reasonable assumption.

3.2.3 Summary of known accuracy and applicability of the RMA-2 model.

The accuracy of two dimensional model codes has already been discussed (see Section 2.2.2.2) and conclusions drawn concerning the scope of previous applications. Norton *et al.* (1973) have compared results from a prototype RMA-2 model formulation to experimental data for vertical flow across a submerged broad crested weir. Discrepancies between observed and predicted data for a series of velocity profiles are given in Table 3.2. These indicate an average discrepancy of ±8.4%, a figure broadly in line with the results of Hervout (1989). The range of these figures is quite large, however it was suggested that this was due to inadequacies in the construction of the finite element mesh in the vicinity of profile E and did not indicate a problem with the model formulation. Examination of water surface profiles showed nearly identical behaviour, with discrepancies of the order of ±3.0 mm for this test problem. The overall accuracy of the RMA-2 model is thus well within the

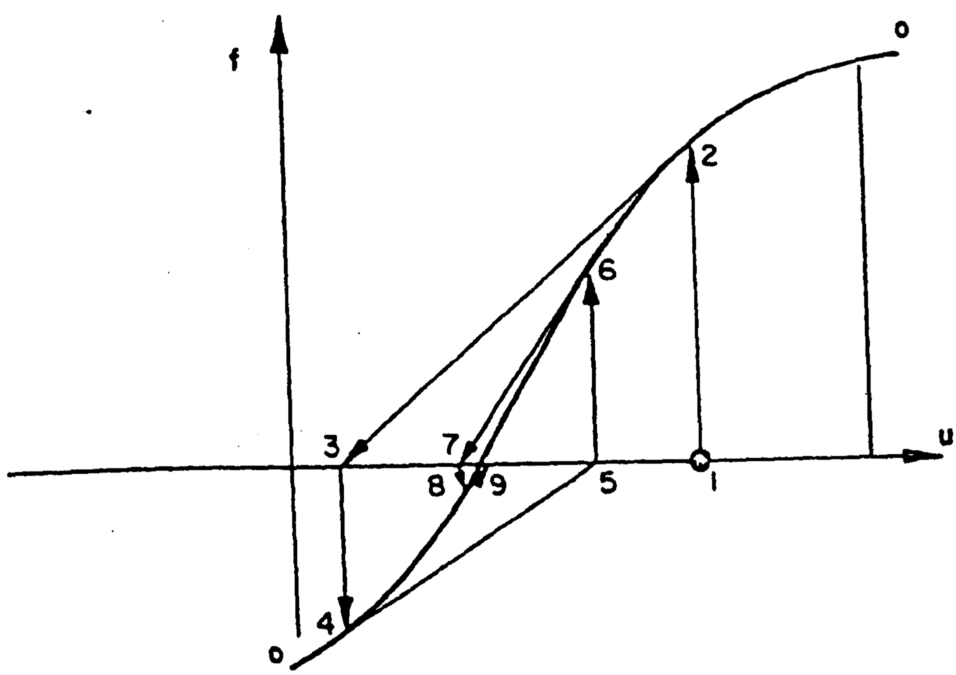


Figure 3.2: Newton-Raphson iterative procedure for a one degree of freedom system. Estimates of the error function f for the variable u are updated at each iteration. Point 1 represents the initial estimate of u, u_1. Point 2 represents the value of the function f for u_1. Point 3 represents the adjusted value for u, u_3 based upon the slope of f at point 2. Thereafter even numbered points represent repetitions of the process occurring at point 2, while odd numbered points represent repetitions of the process occurring at point 3. This refinement continues until convergence is achieved at point 9. The line 0-0 represents the function (u,f). (Taken from Norton *et al.*, 1973)

Distance above flume boundary (ft)	Difference between observed and predicted velocities (ft/sec)	Percentage difference
Profile a		
0.10	0.00	0.0
0.50	0.00	0.0
1.00	0.00	0.0
1.50	0.00	0.0
1.77	0.00	0.0
Profile b		
0.10	0.01	2.5
0.50	0.00	0.0
1.00	0.00	0.0
1.50	0.00	0.0
1.77	0.00	0.0
Profile c		
0.10	0.03	30.0
0.50	0.05	16.7
1.00	0.00	0.0
1.50	0.04	8.9
1.77	0.04	8.9
Profile d		
0.05	0.11	5.9
0.10	0.10	5.3
0.20	0.08	4.1
0.30	0.00	0.0
0.35	0.00	0.0
Profile e		
0.05	0.42	30.0
0.10	0.31	21.4
0.20	0.04	2.6
0.30	0.40	23.5
0.35	0.97	51.1

Table 3.2: A comparison of observed velocities for flow over a submerged broad crested weir with RMA-2 predictions (Norton *et al.*, 1973).

measurement error of data used to parameterize the model and establish solution boundary conditions.

Having demonstrated the basic utility of the prototype model, a variety of applications have been undertaken. Norton *et al.* (1973) have applied the model to reservoir circulation and river confluence studies. These preliminary results confirmed the conclusions drawn with respect to experimental data. Subsequently, applications of the RMA-2 model have been made for such problems as river flow (Gee and Wilcox, 1985), harbours (King and Roig, 1988), tidal marshes (MacArthur *et al.*, 1987) and floodplains (Gee *et al.*, 1990). Figure 3.3a shows a typical finite element mesh for 3.5 km of the upper Mississippi River, taken from Gee and Wilcox (1985). This demonstrates the problem, identified in Section 2.2.2.2, that two dimensional approaches have often failed to resolve river channels separately from the floodplain and that the scale of interest has often been small. More recently, applications incorporating the new approach to wetting and drying of the mesh described in Section 3.2.1.2. have been attempted. A prototype schematization was constructed for a southern section of San Fransisco Bay (King and Roig, 1988) which successfully demonstrated the utility of the approach. This work has now been extended to other flow situations such as floodplains (Gee *et al.*, 1990, see Section 3.2.1.2).

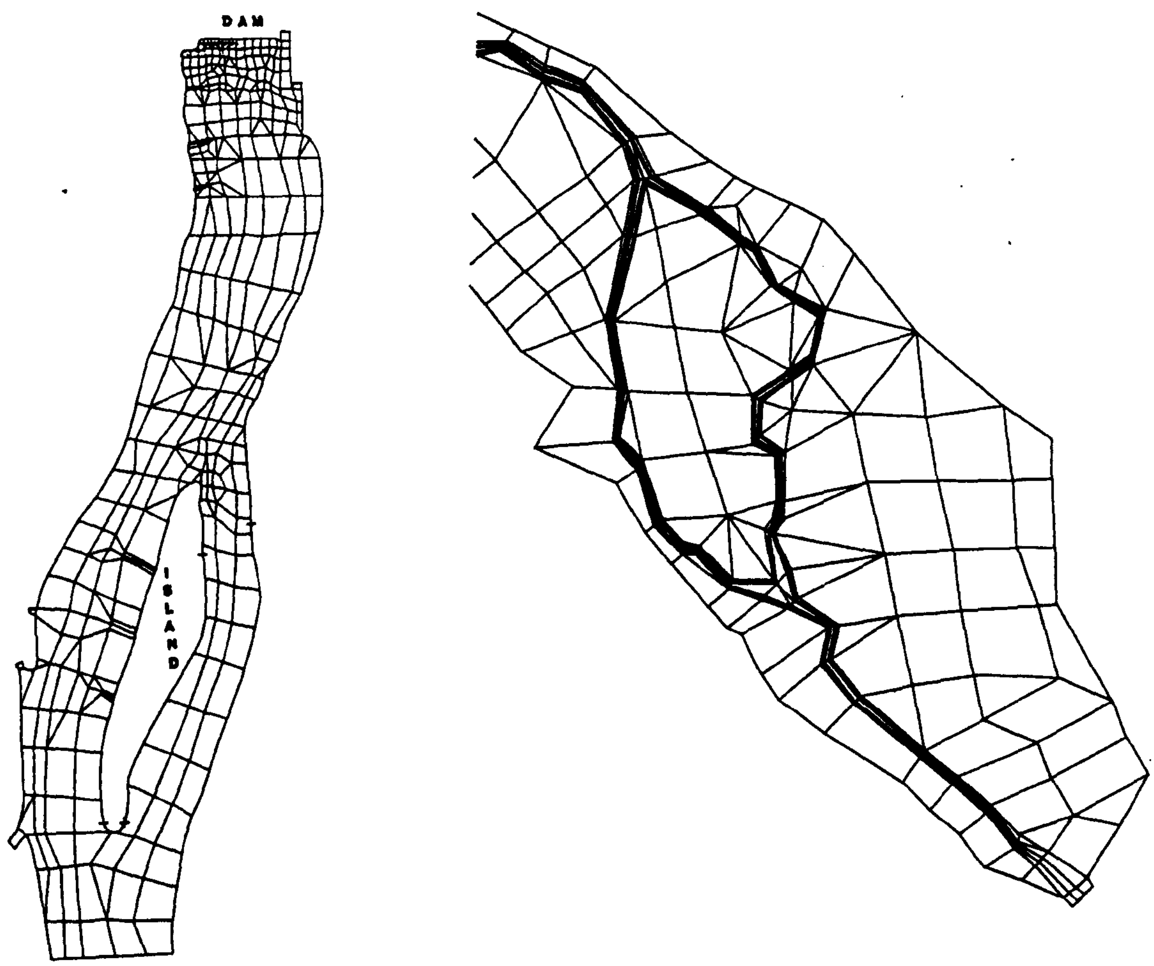
An examination of previous RMA-2 applications confirms the conclusion drawn in Section 2.3 that two dimensional models have only been applied to a restricted set of problems characterised by their scale and by properties of their finite element networks. Despite this, it has been demonstrated (Gee *et al.*, 1990) that RMA-2 provides a sufficiently high level of accuracy and resolution for extension to more complex floodplain flow problems to be considered.

3.3 Model Development for floodplain applications

It has previously been demonstrated that development of the RMA-2 modelling scheme is required to:

i) extend the model to consider river reaches of 10 - 30 km,

ii) develop finite element discretizations capable of resolving river channels meandering within a wider floodplain belt, a b



Scale: 4 cm = 1 km.

Figure 3.3: Typical finite element networks for RMA-2 applications.

a) Small reach scale, channel-only network of the type developed in previous applications.
b) Long reach scale network for compound channels incorporating a seperately resolved channel and floodplains, large scale floodplain elements and complex topography.

iii) develop finite element discretizations capable of incorporating complex topographic features at the long reach scale.

To achieve this, refinement of both the physical model and the topographic discretization has been undertaken. As RMA-2 was developed as a generally applicable two dimensional flow model and has been widely validated, few changes to the formulation of the governing equations were envisaged. The physics of fluid flow are comparatively well understood and can be assumed to hold over the order of magnitude increase in scale of model application proposed in this study. In addition, Chapter 2 has demonstrated that two dimensional models are entirely capable of representing river channel/floodplain flow processes at such a scale. Less well understood is the impact on stability of altering the topographic discretization procedure in order to achieve particular modelling objectives. Both such approaches were therefore followed, with the latter representing a unique means of extending the scope of current hydraulic modelling schemes.

3.3.1 Development of the physical model

Modification of the wetting/drying routine and changes to methods of boundary condition parameterization for floodplain flow applications were necessary in order to improve solution stability for complex mesh networks at the long reach scale.

Modifications to the wetting/drying routine were made to cope with the increased complexity of topographic discretization required for floodplain flow problems. Stability problems were likely to occur in such situations with the prototype model as this allowed elements to be removed from the solution **during** an iteration sequence thus leading to poor convergence. The model code was altered to remove elements only at the **end** of each iteration sequence or time step. In addition, a minor change previously made to bottom elevation was removed as this could cause problems in areas of steep bottom slope.

A change in the determination method for rating curve boundary conditions was also necessitated for compound channel problems. Such boundary conditions provide greatest solution stability and were therefore considered essential for floodplain applications where mesh complexity could introduce additional computational difficulties. The change in geometry occurring at bankfull stage in compound channels and the development of a shear layer between the main channel and floodplain flow, noted in section 2.1.2.1, leads to a discontinuity in the stage-discharge rating curve (see Figure 3.4). Previously, RMA-2 applications have used published historical records to estimate this relationship, as this is relatively accurate for river flow, estuary or harbour problems that consider a single channel. For the out-of-bank portion of rating curves in compound channels such data is frequently unavailable or, if available, subject to measurement errors leading to substantial inaccuracies in discharge estimates (see Table 4.3). Martin and Myers (1991) estimate this to be of the order ±25% if momentum exchange effects alone are unaccounted for. This figure will be further increased by field data errors. Some field estimation procedure for compound channel rating curves that incorporates momentum exchange effects was therefore required if a sufficiently accurate rating curve was not available. A number of approaches to this problem have been reported in the literature (see Shiono and Knight, 1991), from which the HYMO3 modelling package (Williams and Hann, 1973) was selected. This approach takes a surveyed cross section and calculates flow at a series of elevations using the Manning equation. Momentum exchange effects have been incorporated (Baird and Anderson, 1990), based on an analytical approach devised by Knight and Hamed (1984) from empirical data. This discharge/elevation data can then be approximated to the required rating curve form by regression techniques. More complex methods of rating curve estimation (e.g. Ervine and Baird, 1982; Wormleaton, 1988; Shiono and Knight, 1988; Wark *et al.*, 1990) were considered, however the HYMO3 approach is particularly suitable for use in conjunction with RMA-2 as it has parsimonious data requirements (namely a value for the Manning's *n* coefficient) already available from the RMA-2 initialization, it is physically based and it incorporates all significant processes. In addition, it is inappropriate to implement a complex procedure before simple approaches have been sufficiently evaluated. This evaluation is therefore undertaken in Section 4.2.1.4.

3.3.2 Development of the physical representation by finite elements

The topographic discretization developed here for use in long reach floodplain applications is significantly different to that used in previous studies (see Figure 3.3). **This therefore represents a major area of originality in this study.** Three unique developments were incorporated into the topographic discretization.

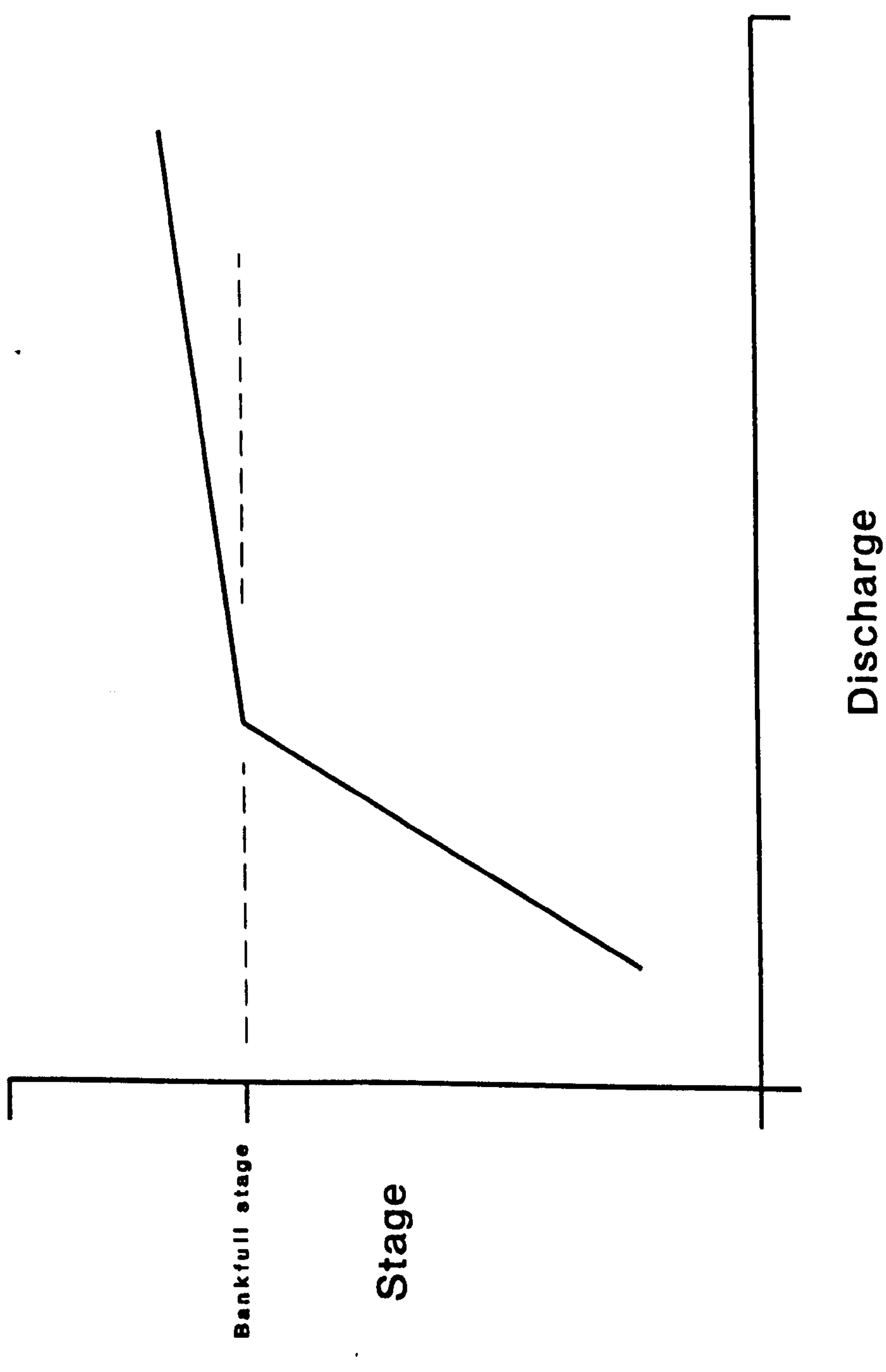


Figure 3.4: Typical stage-discharge rating curve for compound channels showing discontinuity at bankfull discharge.

Firstly, a meandering channel was included within the finite element network. This was resolved separately from, but continuous with, the floodplain (see Figure 3.3b), allowing two dimensional aspects of meandering compound channel flow to be simulated in a realistic fashion and incorporating the momentum transfer mechanism identified in Section 2.1.1. An initial prototype study by Gee *et al.* (1990) used a triangular channel cross section in the topographic discretization. This produced unrealistically high channel velocity vectors and was consequently rejected. However, over-enhancement of channel representation can introduce additional complexity to the finite element network, leading to lower stability and reduced computational efficiency. A trapezoidal channel geometry was therefore selected as this represents the next most complex discretization, using four nodes to represent the cross section instead of three. In addition, prismatic channel cross sections were found to create sharp, and potentially unstable, breaks of slope in the finite element mesh and a poor distribution of flow across the channel. A non-prismatic trapezoidal channel discretization was therefore implemented (see Figure 3.5).

Secondly, the scale of elements used to represent the physical system was necessarily increased for long reach applications. This allowed an approximate order of magnitude increase in the scale of reach length that could be effectively represented, while maintaining reasonable computing requirements. The number of nodes and elements in any particular network was therefore kept relatively constant. However their maximum size was increased to approximately 200 x 200 m. This represents an increase in maximum element size over previous two dimensional finite element models of between 100 - 500% although much smaller elements can still be used to represent channel geometry and complex floodplain topography. Despite the size of such elements they were considered appropriate to floodplain modelling due to the low gradients of system state variables present in such flows. In addition, a concern with mesoscale (10 - 100 m) rather than microscale (1 - 10 m) flow patterns allows this scale of representation to effectively resolve all flow features of interest.

Thirdly, mesh complexity for long reach floodplain applications was increased. The ability of finite element networks to incorporate topographic complexity is well documented, however related stability issues have only been resolved for small scale applications. Physical features such as embankments, floodplain constrictions, buildings and channel bifurcations encountered in long reach applications have not previously been included in finite element discretizations. Finite element

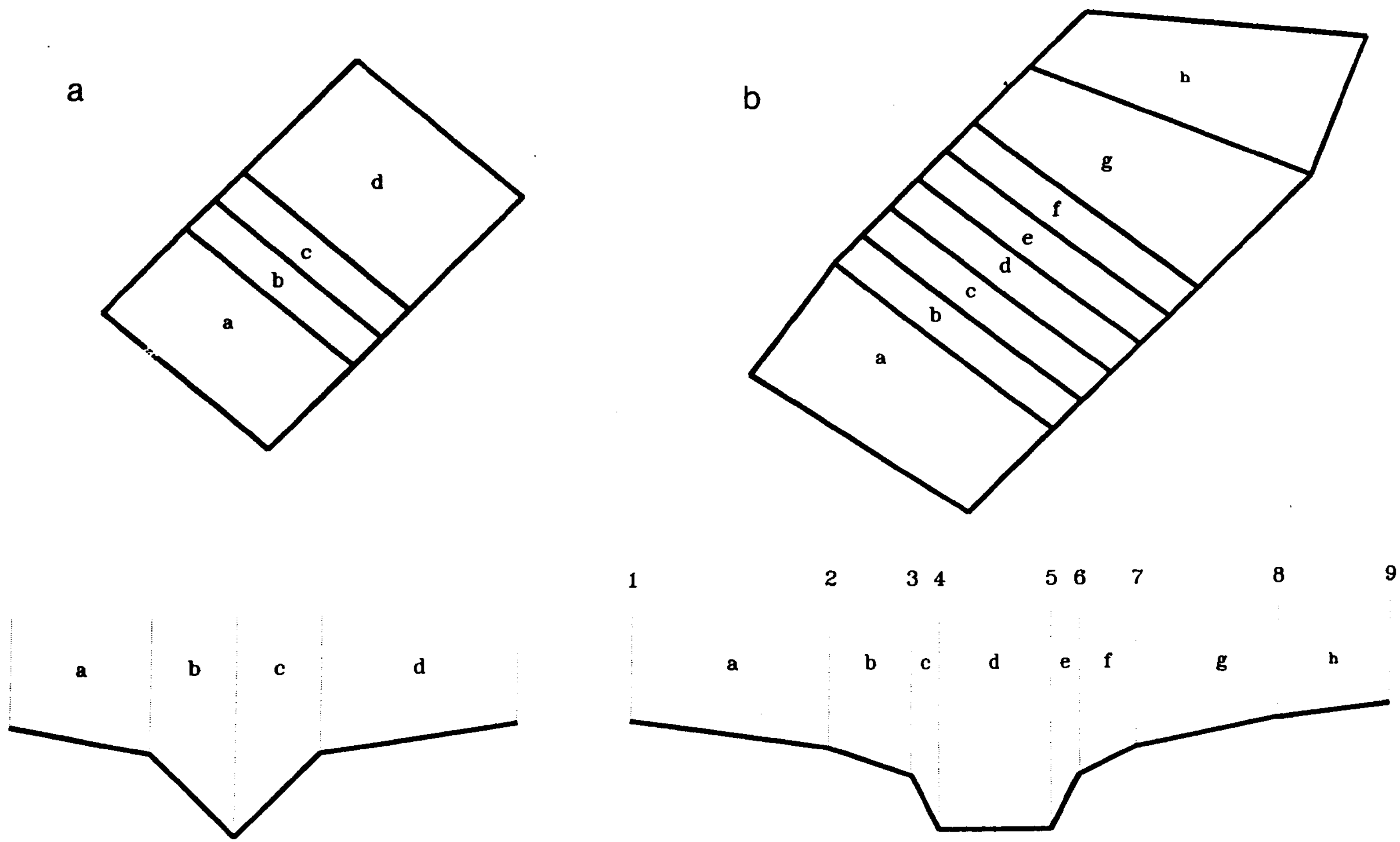


Figure 3.5: Conceptual mesh cross sections for RMA-2 applications.

a) Prismatic triangular cross section used by Gee *et al.*(1990). This produced unrealistically high channel velocities.
b) Non-prismatic trapezoidal cross section developed for the refined RMA-2 model proposed here.

discretizations were therefore developed for this project that represent such complexity at long reach scales. In addition, modelling of topographically simple reaches does not allow definition of the model's operational limits for floodplain applications. Inclusion of complex topographic features into the finite element discretization and evaluation of these developments will thus allow this to take place. This is addressed in Chapter 5 along with a resolution of stability issues relating to the inclusion of long reach scale topographic complexity.

3.3.3 Summary

A series of developments to the RMA-2 model have been undertaken to enable its extension to consider the flow problem outlined in Chapter 1. This has entailed modification of the model code, parameterization procedures and, most significantly, representation of the physical environment by the finite element network. The enhancements in modelling capability over the prototype RMA-2 scheme resulting from these developments are summarised in Table 3.3. **This modelling strategy therefore differs significantly from currently available hydraulic models in its ability to simulate river channel/floodplain flow over long reach lengths at a high level of resolution. Previous models are only suitable for more restricted areas of investigation.**

3.4 Research design

The structured testing, development of operational rules for and validation of the modelling strategy developed in this Chapter is therefore the main task of the remainder of this thesis. It is now appropriate to develop a research design to achieve this.

This work will follow the three stage model evaluation strategy outlined by Sargent (1982) and summarised in Figure 3.6. Model evaluation consists of:

i) Mathematical model validation (Chapter 4) - to determine whether the specified conceptual model is theoretically correct and suitable for its intended use.

Application/development	Prototype RMA-2	Enhanced RMA-2 (This thesis)
Scale	0.5 - 2 km	10 - 30 km
Cross sectional representation	channel only	channel and floodplain continuum
Wetting/drying algorithm stable for long reach scales	no	yes
Parameterised on the basis of ...	data collection programmes	available data
Incorporation of complex topography	only at small scale	at all scales

Table 3.3: Summary of developmental advances over the prototype RMA-2 model, implemented during this project, which allow the high resolution simulation of river channel/floodplain flow at the long reach scale.

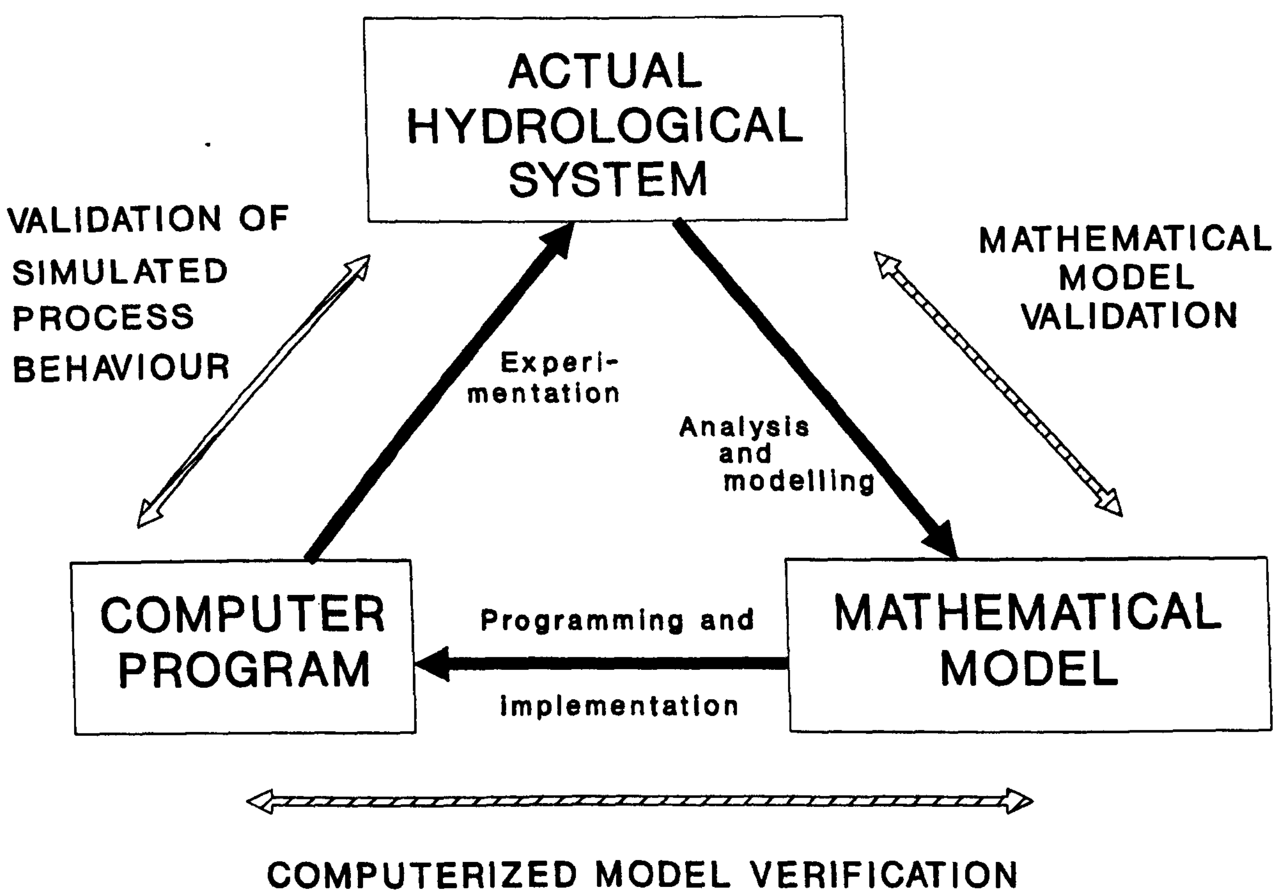


Figure 3.6: A three stage model evaluation strategy (after Sargent, 1982).

ii) Computerized model verification (Chapter 4) - to ensure the conceptual model has been correctly transferred into computer code and the whether numerical scheme selected is able to provide a stable and accurate approximation to the true solution. Computerized model verification therefore involves:

a) testing output sensitivity to variability in model inputs,
b) examination of solution convergence and continuity,
c) model simulations under abnormal conditions to fully test for areas of model breakdown.

No published results exist for sensitivity testing or abnormal input simulations for the prototype RMA-2 model. Previous research has largely been confined to the validation of simulated process behaviour. **This aspect of the research design will therefore provide the first complete computer model verification of RMA-2.**

iii) Validation of simulated process behaviour (Chapters 5 and 6) - to determine whether the implemented model provides an adequate representation of reality. Model predictions will be compared with observed process behaviour for a range of flow conditions. This will allow an assessment to be made concerning fulfilment of the research objectives stated in Chapter 1 and provide an indication of appropriate future research directions. In addition, objective comparisons can be made with alternative modelling schemes.

Validation of simulated process behaviour for the refined RMA-2 model will be carried out on a single test reach due to the time required to implement complex finite element networks and develop these to an operational condition. It is acknowledged, however, that the use of a single study reach must mean that conclusions drawn from the evaluation procedure, although appropriate to other floodplain environments with similar characteristics, must be interpreted in this context.

A two phase evaluation strategy will be used to assess the model simulations developed for this test reach. As a first step, the scheme's ability to produce results at the downstream outlet of the modelled system will be assessed (Chapter 5). Having satisfactorily established the limits of model performance in this context, further work will concentrate on validating model predictions of process behaviour over the whole reach (Chapter 6).

Finally, on the basis of conclusions drawn from this evaluation process (Chapter 7), a number of extensions to the modelling capability are explored (Chapter 8). This research design is summarised in Figure 3.7 and provides the basis for the work reported in the following Chapters.

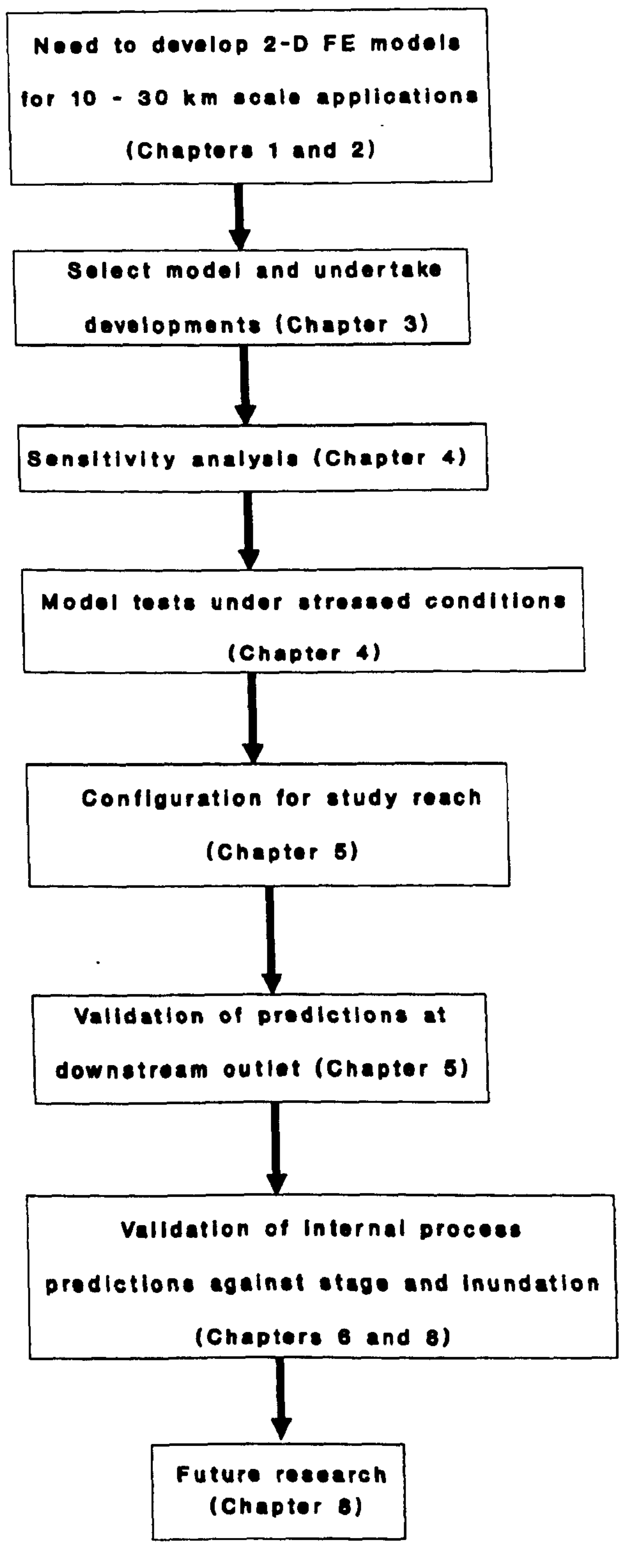


Figure 3.7: A research design for the evaluation of the refined RMA-2 modelling scheme.

II Model evaluation

CHAPTER 4

Initial model evaluation

The research design developed in Chapter 3 has identified a three stage model evaluation strategy for the refined RMA-2 model. This Chapter covers the first two stages of this process, namely mathematical model validation and computer model verification. These are undertaken prior to model implementation on specific test reaches and provide feedback for the model development process.

4.1 Mathematical model evaluation

Mathematical model evaluation assesses whether the simplifying assumptions made by the model are valid. This is a discursive method designed to be applied at an early stage of model specification and development (Howes and Anderson, 1988) and ensures that system behaviour is adequately represented for the applications and scales under consideration. The aim of this study is the simulation of flood inundation over long reach scales. Therefore, an assessment will be made of the specified model's ability to achieve this. Model assumptions can be divided into two areas; those relating to the physical model and those inherent in the numerical solution scheme.

4.1.1 Assumptions of the physical model

4.1.1.1 Assumptions of the governing equations

Section 3.2.1.1 defined the major assumptions of the Reynolds form of the depth averaged Navier-Stokes equations. These were:

i) the fluid is incompressible and of constant density,

ii) vertical velocity and acceleration are negligible,

iii) the river bed does not change with time.

For incompressible flow it is assumed that variations in the pressure field of the fluid do not produce any significant variation in density. Density is therefore constant within the fluid body. Incompressibility is a reasonable assumption for liquids such as water that have a high bulk modulus. As a result density variations are small even in the presence of large pressure changes (Tritton, 1977). For flow in natural open channels, pressure variations (atmospheric pressure, pressure due to water depth) are small and the condition of incompressibility is maintained. This assumption then allows simplification of the controlling equations. Density changes due to solutes in fresh water are negligible due to their low and relatively uniform concentration levels. Variations in suspended sediment concentration do occur, but again at densities too small to be significant in terms of the overall density field of the flow. Constant density also implies that the fluid is isothermal. This would appear to be a reasonable assumption due to the shallowness of the water body and the effects of turbulent mixing processes.

The assumption that vertical velocity and acceleration are negligible simplifies the three dimensional flow field observed for meandering compound channels (see Section 2.1). Thus, velocity will only vary in the x and y cartesian planes, with any variation in the z plane being averaged over flow depth. The governing equations therefore simulate a depth averaged two dimensional approximation to a three dimensional flow problem. The meandering compound channel flow features which a simulation model for flood inundation must incorporate have been outlined in Section 2.1.3. These are the momentum transfer mechanism between main channel and floodplain flows and the spillage onto the floodplain of water delivered by the channel and its subsequent return, again with an associated momentum transfer. Incorporation of these mechanisms into the model will therefore allow accurate simulation of flood inundation extent. As such mechanisms primarily involve longitudinal and lateral transfers of fluid mass and energy, depth averaged flow is an appropriate assumption. This mathematical description will necessarily exclude certain processes, such as secondary flow and horizontal shearing (Kiely, 1990), but will be sufficiently detailed to permit simulation of the bulk flow characteristics.

The governing equations also assume that the river bed does not change with time. Thus the effect of erosion and deposition processes on the channel bed topography is not sufficient to cause variations in the flow field. Erosion and deposition processes in temperate alluvial channels primarily involve transfers of suspended sediment, with typical concentrations less than 1000 mgl^{-1} (Bates *et al.*, 1992). Such process rates are consistent with the assumption of an unvarying river bed over an RMA-2 simulation event (a single flood). Channel change sufficient to cause variations in the flow field does occur in such systems, but this is usually a catastrophic phenomena confined to specific high magnitude events. For the majority of flood events simulated by the RMA-2 model, the above assumption will therefore hold. However, for an event where significant channel change occurs the application of RMA-2 may be problematic.

4.1.1.2 Assumptions of the wetting and drying routine

The wetting and drying routine of the RMA-2 model described in Section 3.2.1.2 is based on two assumptions, one concerning the dynamics of the inundation process and one concerning the simulation method. Inundation is assumed to occur by smooth lateral movement of the flow boundary away from the main channel. This will provide a good approximation to the physical system in most cases. Problems may occur when the main channel is not the sole source of floodwater delivered to the floodplain. For example, drainage ditches may deliver water to the back of the floodplain and thus create a two sided inundation front. Additionally, the return of water to the main channel may be complicated in some floodplain systems by this local pattern of drainage and by water going into storage. The assumption of a horizontally moving flow boundary will fail to capture such complex behaviour and the model would prove inappropriate for situations where such behaviour dominates the inundation process. However, the conceptual model of inundation assumed by RMA-2 is adequate for a large number of floodplain systems and represents a much better approximation to physical reality than the irregular flow boundary assumed by alternative two dimensional simulation models (see for example Nece and Falconer, 1989).

In approximating a horizontally moving flow boundary, RMA-2 maintains a small water depth over dewatering elements as long as they remain in the solution (see Section 3.2.1.2). This approximation assumes that this volume of water is negligible. This is a reasonable assumption when considered relative to typical flood volumes and introduces smaller mass discontinuities than alternative element elimination methods (King and Roig, 1988).

4.1.1.3 Assumptions of model parameterization

Three main assumptions are inherent in the variables used to parameterize the governing equations. These are:

i) boundary friction for an element can be represented by a single friction factor value,

ii) the Boussinesq approximation to the turbulent Reynolds stress holds true,

iii) the specified boundary conditions adequately define the physical system.

It is common practice in hydraulic models, such as RMA-2, to define boundary friction in terms of a single friction factor coefficient such as Manning's n or Chezy's C. These are defined intuitively from experience or by reference to some classification scheme, such as Chow's (1959) photographic interpretation method for Manning's n. Two problems arise with this; firstly the value of the friction coefficient is, in reality, not a constant and secondly all evaluation methods incorporate a degree of subjectivity. It has been demonstrated (Section 2.1.1) that boundary friction varies with flow depth, vegetal effects and changes in channel bedform. It has also been demonstrated that boundary friction shows a high degree of spatial heterogeneity. A unitary value of boundary friction is therefore an abstraction of a much more complex physical reality. However, this variation mostly occurs on time and length scales that are orders of magnitude less than the resolution of the bulk flow simulations being attempted in this study. For example, spatial heterogeneities in boundary friction due to vegetal effects occur over scales of 10 - 100 cm. This is in contrast to the 10 - 100 m resolution of flow field properties provided by the enhanced RMA-2 model. Inclusion of such detail in the model formulation can thus be reasonably regarded as unnecessary. A single boundary friction value is therefore sufficient to simulate all flow properties of interest at this scale. Current engineering practices that counter subjectivities in the estimation procedure rely on the experience of the operator or the careful application of known evaluation schemes. These would appear sufficient to allow the friction factor to be assessed in a relatively objective manner. Thus the assumption of a single valued friction coefficient contains flaws but is appropriate for bulk flow simulations if applied with care.

The Boussinesq approximation (see Massey, 1983 p.150) provides a very simple first order representation of the turbulent Reynolds stresses. Although more complex turbulence models exist it can be argued, as in the case of boundary friction, that the Boussinesq approximation provides an adequate representation of the turbulence structure given the time and length scales of the flows under consideration. The fine scale structure of turbulence in open channels is not significant in terms of the simulation of the bulk flow characteristics of flood inundation. Therefore a relatively coarse turbulence model, such as that provided by the Boussinesq approximation, is sufficient to simulate all the flow properties of interest. More complex turbulence models would cause the proposed model to be overspecified in this respect.

If the specified boundary conditions are not sufficiently accurate to define the physical system, the proposed model will be invalidated. In general, boundary condition accuracy will vary significantly between systems. It is therefore necessary to assess this assumption in the context of each specific application.

4.1.2 Assumptions of the numerical solution scheme

The numerical solution scheme outlined in Section 3.2.2 incorporates the following assumptions:

i) the integral evaluated over an entire element by the interpolation function is equal to the summation of the integrals performed over element subregions (Huyakorn and Pinder, 1983),

ii) the solution found by the iterative procedure adequately approximates to the true value,

iii) in the implicit finite difference scheme used for time dependent solutions, acceleration is assumed to vary linearly over a time interval σ, where $\sigma = \theta\Delta t$ and $\theta > 1.37$.

Assumption (i) ensures the interpolation functions are an adequate approximation to the variation of the governing equations over each element. The form of the interpolation function must be of an order (linear, quadratic, polynomial etc.) sufficient to uniquely determine the variation in dependent variables over an element. RMA-2 assumes that changes in the velocity field can be approximated by a quadratic function. This obviously provides a better approximation to a non-linear system of equations than a linear interpolation function, however the case for using a higher order polynomial as an interpolation function is not sufficiently borne out by physical evidence. When considering gradually varying unsteady flow problems such as the passage of a flood wave, variation in velocity and stage is gradual and damped. It is therefore reasonable to assume that a quadratic function is sufficient to account for this variation.

The accuracy of iterative procedures for non-linear systems of partial differential equations is impossible to determine mathematically (Pinder and Gray, 1977). Rates of convergence can be calculated, however it is impossible to determine whether this approximates to the true solution or not, this must be assumed. However, studies which have compared two dimensional model code results to experimental data (Norton *et al.*, 1973; Hervout, 1989) indicate, albeit indirectly, the accuracy of iterative solution methods. The assumption that an iterative solution approximates well to the true solution can therefore be considered valid.

The assumption of linear acceleration over the time interval, σ, has already been discussed in Section 3.2.2. It was concluded that such an assumption is reasonable for problems involving gradually varied, unsteady flow of the type being considered here.

4.1.3 Summary of mathematical model evaluation

The assumptions inherent in the physical model and its numerical solution scheme have been evaluated in the context of simulating the bulk flow characteristics of flood inundation at long reach scales. **It has been demonstrated that the inherent assumptions do not compromise this objective. We may therefore state that the**

mathematical model specified in Chapter 3 is valid for the simulation of this flow problem.

4.2 Computerized model verification

This second stage of model evaluation ensures the mathematical model identified in Chapter 3 and mathematically validated in Section 4.1 has been correctly transferred into computer code and that the numerical scheme selected is able to provide a stable and accurate approximation to the true solution. This involves a series of tests that establish whether the computer model simulates those processes expected of it and that those processes interact in a logical way, consistent with the mathematical model. This Section therefore examines three areas of model behaviour to establish the veracity of the computer model; model sensitivity to parameter variability, stability of the numerical solution scheme and model response to abnormal parameter values. This will provide the first complete computer model verification of RMA-2.

4.2.1 Sensitivity analysis

The aim of this aspect of computerized model verification is to determine model response to small variations in the normal range of input parameters. In terms of model verification, this will determine whether the computer model is responding in a manner consistent with the specified mathematical model and with the known behaviour of the physical system. The results of the complete first pass analysis presented here will allow isolation of significant variables for further testing in the context of specific applications. It will also allow resolution of a number of issues relating to model implementation, such as data provision and mesh construction. The data set produced by the analysis has a number of other attributes. The sensitivity analysis results can be used to quantify the effect of uncertainty in parameter estimation (see Table 3.3) on model results. In addition, abnormalities in the normal range of model behaviour can be identified and the data set produced can be a significant input to model calibration.

Some sensitivity testing of the refined RMA-2 model has already been reported (Baird and Anderson, 1990). In particular, the sensitivity of specific flow (the product of stage and velocity) to variation in boundary friction was assessed. It was found that changes in boundary friction could generate significant changes in specific flow and it was concluded that boundary friction, in terms of both its magnitude and spatial distribution, was a powerful calibration parameter for the RMA-2 scheme. Apart from this limited analysis no other published results exist concerning sensitivity analysis of RMA-2 for floodplain, or indeed other, applications.

4.2.1.1 Methodology and experimental design

Howes and Anderson (1988) outline two possible methodologies for constructing a sensitivity analysis research design; a deterministic approach or a stochastic one. The former method considers the effect of a small change in the value of a parameter or group of parameters on model output. McCuen (1973) suggests two possible methods of analysis:

i) differentiation of the governing equations with respect to the factor, f_i,

ii) factor perturbation, where the factor, or groups of factors, are incremented by a small amount and the change in model output computed.

Of these, factor perturbation is the most commonly used approach as the mathematical basis of the differentiation method has not been sufficiently developed (Howes, 1985). Stochastic sensitivity analysis is a more robust, but computationally more intensive, methodology. Here input parameters are randomly selected from probability distributions covering the range of physically realistic parameter values. This allows model sensitivity to be analysed against a much wider range of data uncertainty (Howes and Anderson, 1988) and reduces the possibility of abnormal results skewing the sensitivity analysis data set. However, to obtain a statistically valid result upwards of 25 stochastic runs per parameter combination must be considered.

For this first pass analysis all relevant model parameters were considered. These can broadly be divided into three categories as follows:

I) mathematical model parameters,

- i) eddy viscosity coefficient,
- ii) boundary friction coefficient,

iii) domain coefficient depth range,
iv) minimum domain coefficient depth range,
v) time step,

II) mesh construction parameters,

vi) longitudinal slope,
vii) lateral slope,
viii) channel shape,
ix) channel sinuosity,
x) mesh resolution,

III) downstream boundary condition,

xi) stage discharge rating curve.

In order to simplify the sensitivity analysis and reduce computing time, the upstream boundary condition (an inflow rate) was not included, as the likely impact of this parameter was felt to be already well understood. In addition, estimation errors for this parameter are low and cannot, unlike the downstream boundary condition, be affected by any modelling decisions made by the user. Less well understood was the impact of abnormal, in particular low, inflow rates. This is therefore considered in detail in Section 4.2.3.

The sensitivity of these parameters was then tested for a variety of flood events and test reaches. In order to ensure a comprehensive analysis, two experimental floodplain reaches of varying complexity[1] (see Figures 4.1) were constructed and three floods of varying magnitude[2] (Figure 4.2) simulated, with three or five values of each parameter being tested. Mathematical model and boundary condition parameters were simulated with a range of five values, whilst mesh construction parameters were simulated for three conditions due to the difficulty of producing meaningful differences in mesh geometry that could be objectively analysed. For this reason also the testing of mesh construction parameters was confined to the simpler of the two test reaches; test reach 1 (Figure 4.1a). A programme of dynamic model tests was chosen in preference to steady state simulations as a primary research objective

1. The experimental floodplain reaches used were derived from sections isolated from the finite element network constructed to simulate the River Culm, Devon, UK. This application is discussed in Chapter 5. The sections are therefore an order of magnitude smaller than those which the refined model will deal with in order to reduce computing requirements, but are obviously of a comparable resolution.

2. The flood hydrographs simulated are based on observed data for the Woodmill gauging station on the River Culm and correspond to events with recurrence intervals of 1, 10 and 100 years respectively.

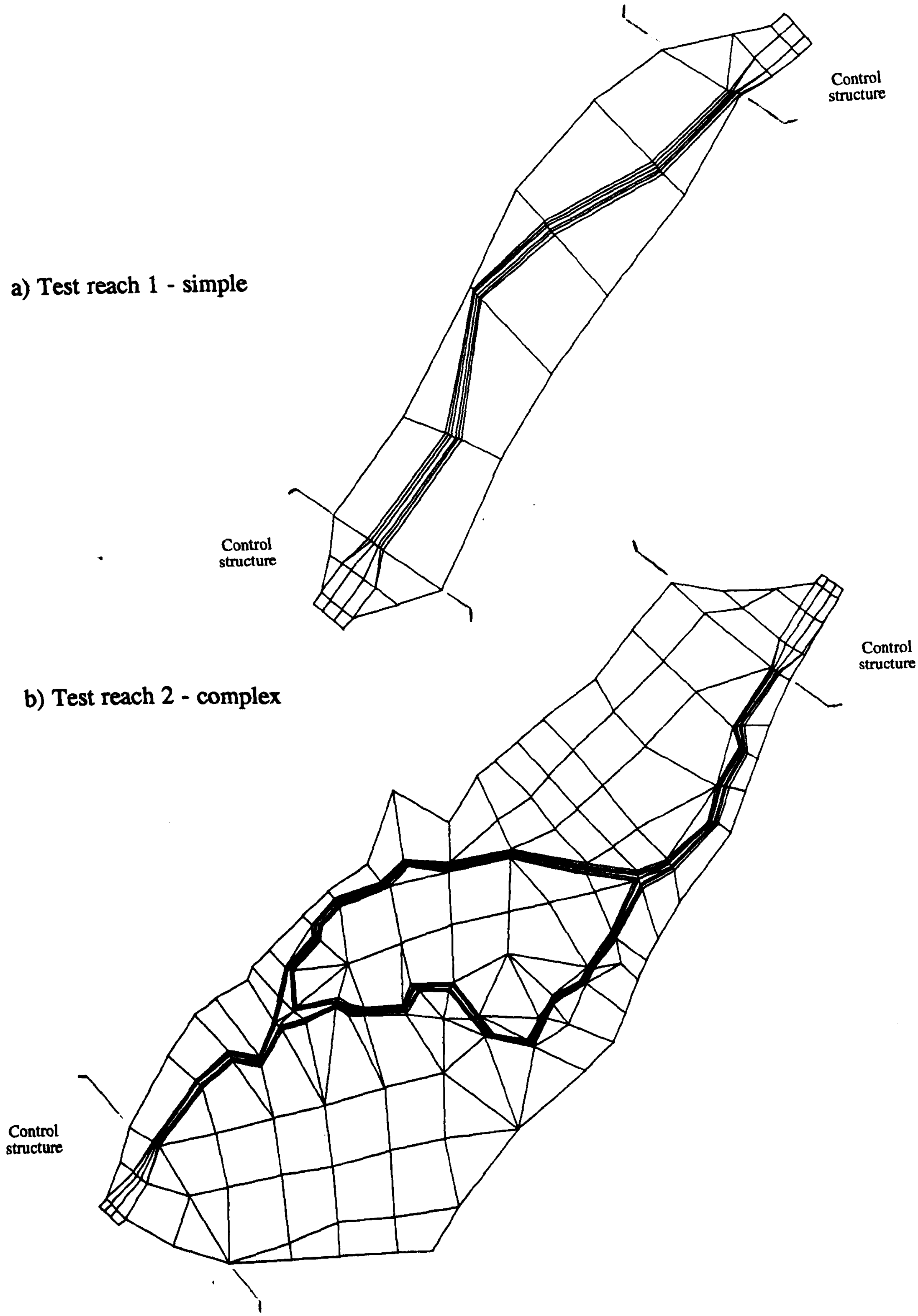


Figure 4.1: Experimental test reaches used in the RMA-2 sensitivity analysis. a) Simple, b) complex.

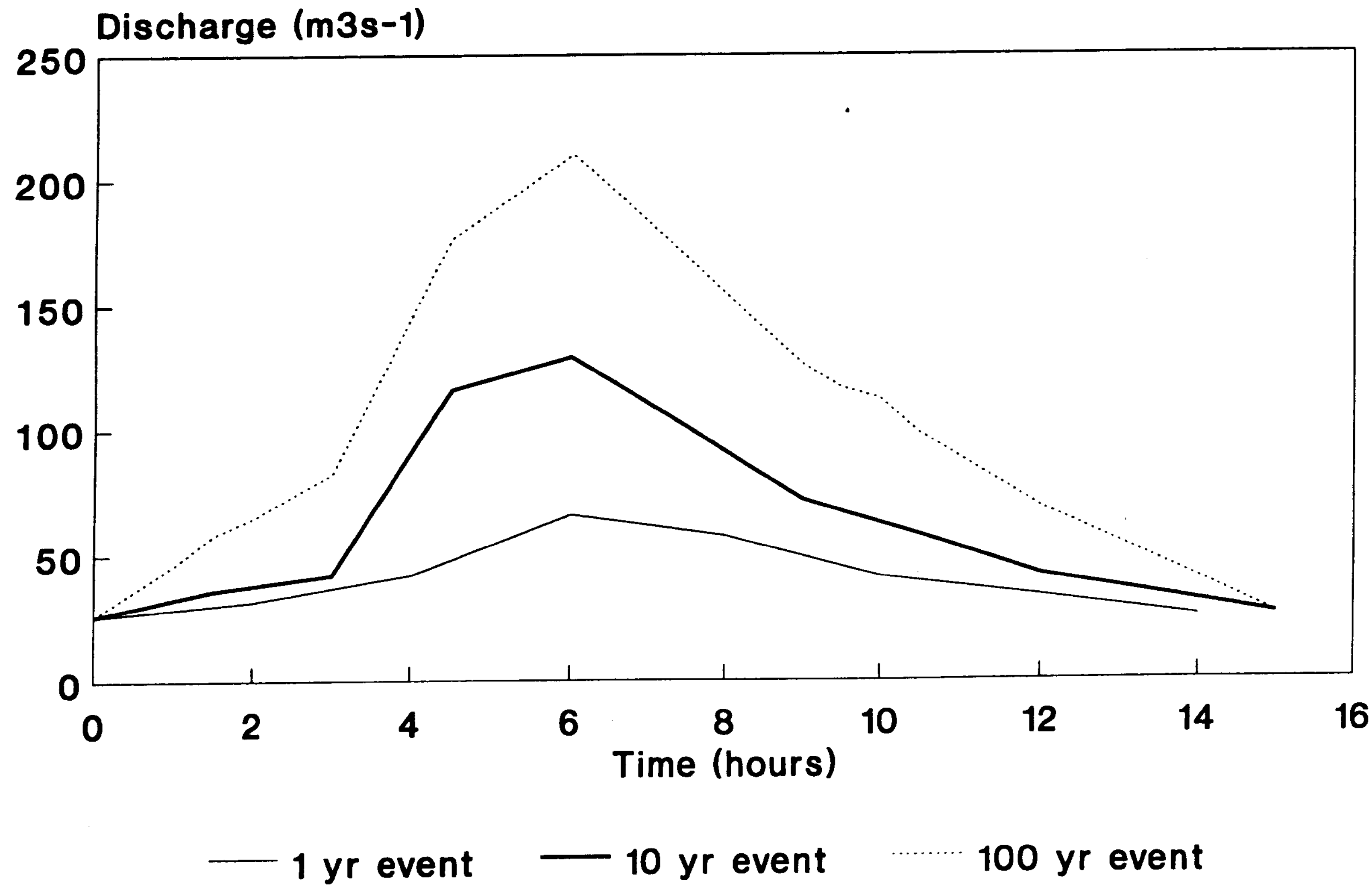


Figure 4.2: Experimental flood events used in the RMA-2 sensitivity analysis.

identified in Chapter 1 was to use the RMA-2 scheme to simulate whole flood events. It was therefore considered essential to perform sensitivity testing under the same dynamic conditions, if results were to be meaningful in the context of potential applications. This experimental design is summarised in Figure 4.3.

In performing this analysis, computer time became a highly limiting factor, with each simulation requiring up to 15 minutes of c.p.u. time on a MIPS M/2000 machine. For this reason a deterministic single factor perturbation analysis, holding all other factors constant, was selected as the only acceptable methodology. Stochastic and combinatorial deterministic analyses would require 7125 and 3.8 x 10^{73} simulation runs respectively. The chosen methodology reduces this to a requirement of 285 runs, equivalent to 3 months of real time computer simulation.

4.2.1.2 Sensitivity analysis results

Physically realistic initial values of model parameters were selected (see Section 5.2.1) and a reference simulation calculated for each recurrence interval event. The result of incrementing model parameters was then assessed as percentage change in peak velocity and stage, recorded for a number of channel and floodplain nodes at the downstream extremity of each experimental reach. The change in time to peak floodplain stage was also recorded. It was felt that assessing sensitivity analysis results in terms of the dependent variables would allow the precise hydraulic effects of parameter manipulation to be more easily disaggregated. Furthermore, evaluating velocity results for both channel and floodplain nodes would allow the impact of factor perturbation on momentum transfer to be examined. Finally, such a design for interrogation of the results files also meant that complex model output could be effectively summarised.

Sensitivity analysis results are summarised for the six most sensitive parameters in Figures 4.4 - 4.9. Results have also been summarised in terms of their unit sensitivities, given for this particular analysis by:

$$US_f = \frac{\Sigma \left[\dfrac{\Delta F_o}{|\Delta F_i|} \right]}{n} \times 100 \qquad (4.1)$$

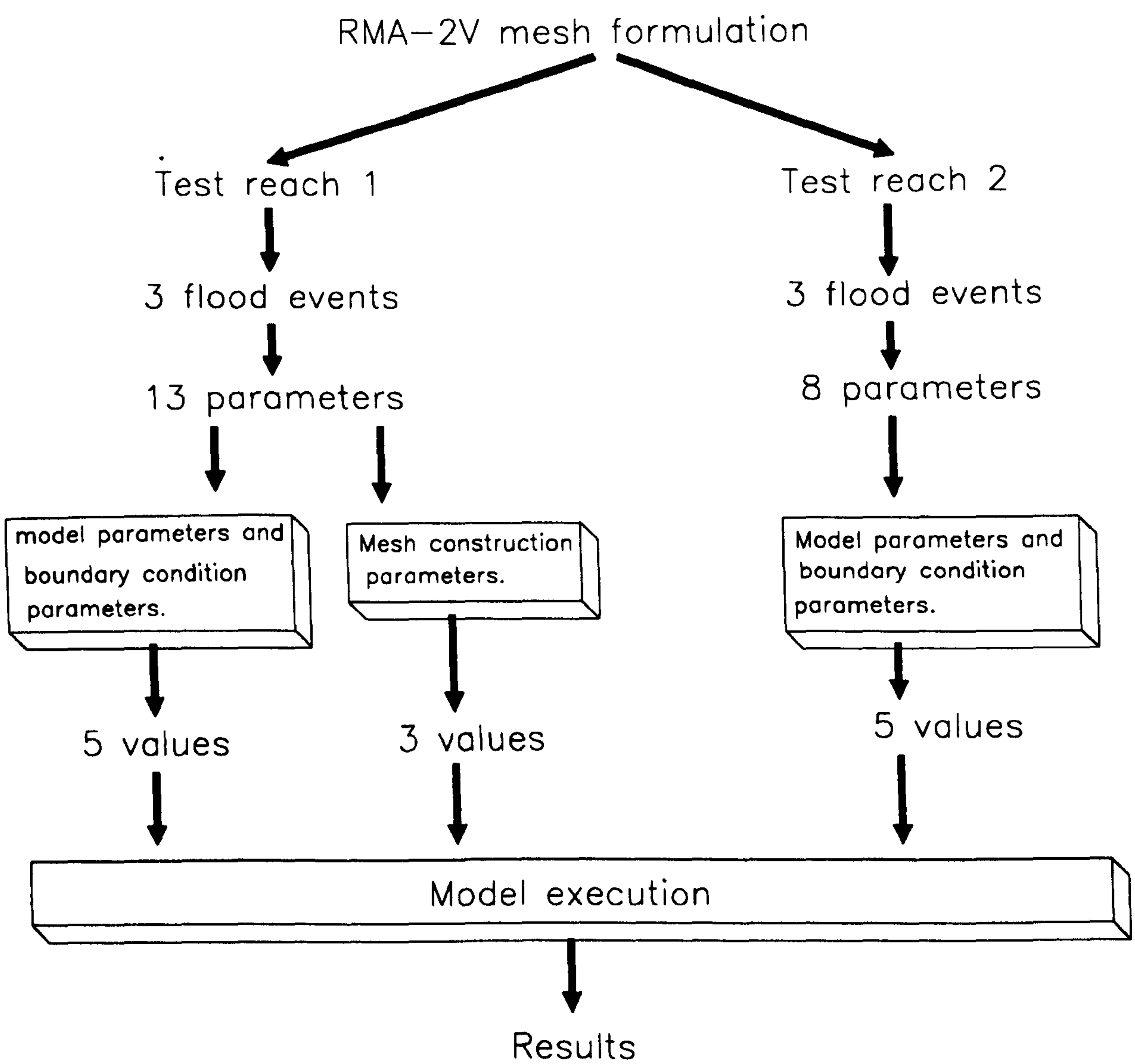


Figure 4.3: Experimental design for a first pass sensitivity analysis of the refined RMA-2 model.

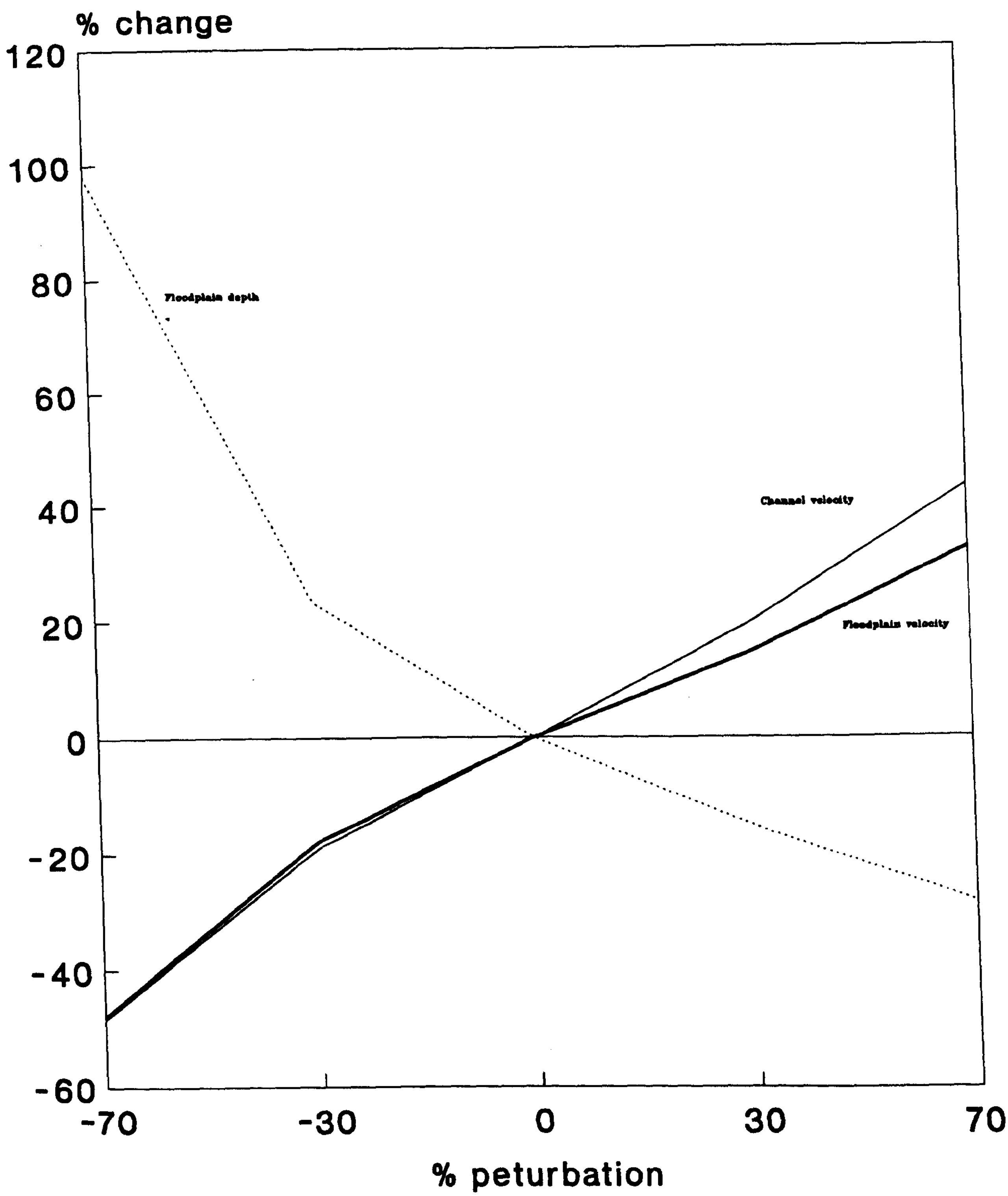


Figure 4.4: Summary of sensitivity analysis results for the stage discharge rating curve forming the downstream boundary condition of RMA-2.

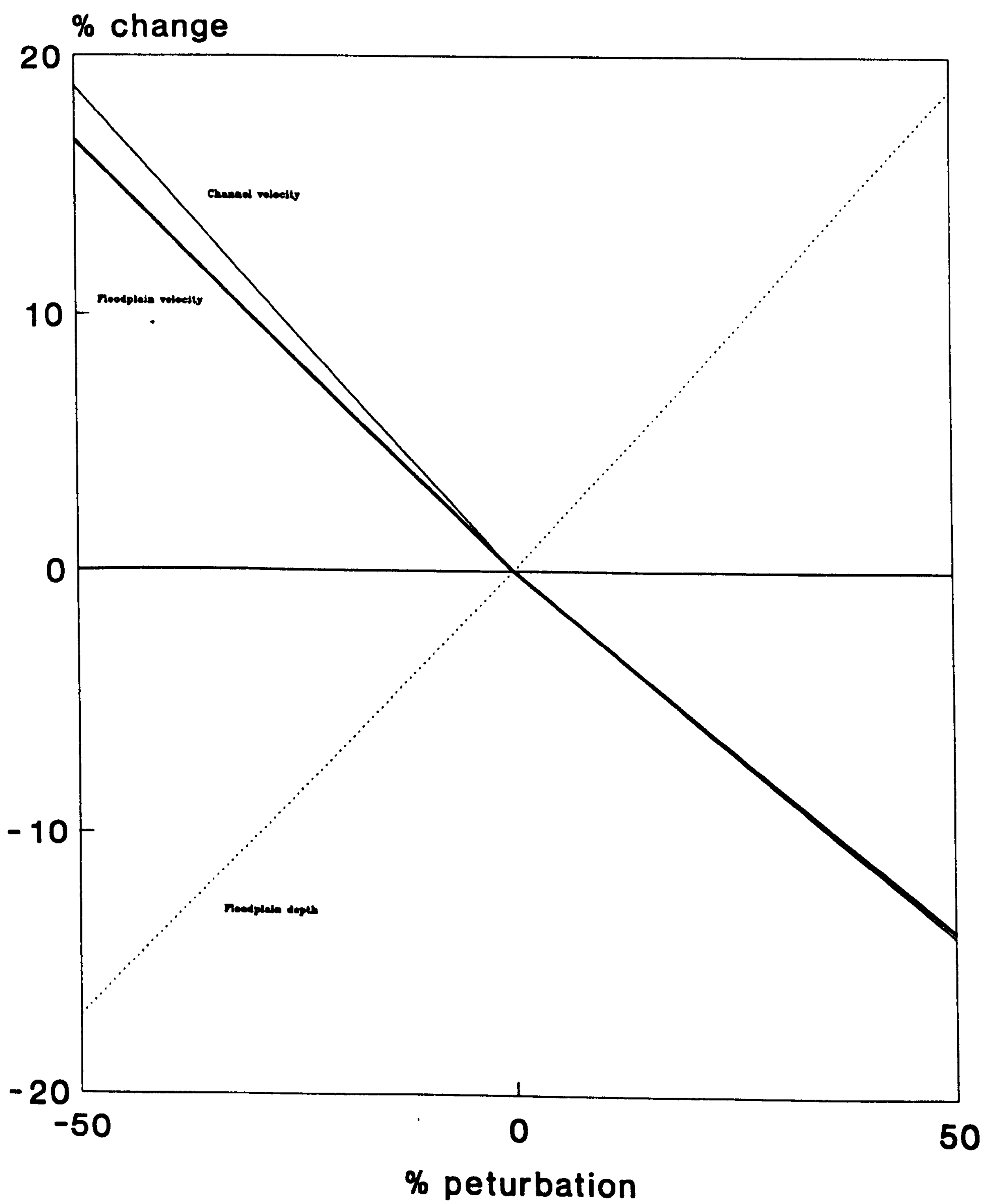


Figure 4.5: Summary of sensitivity analysis results for mesh longitudinal slope.

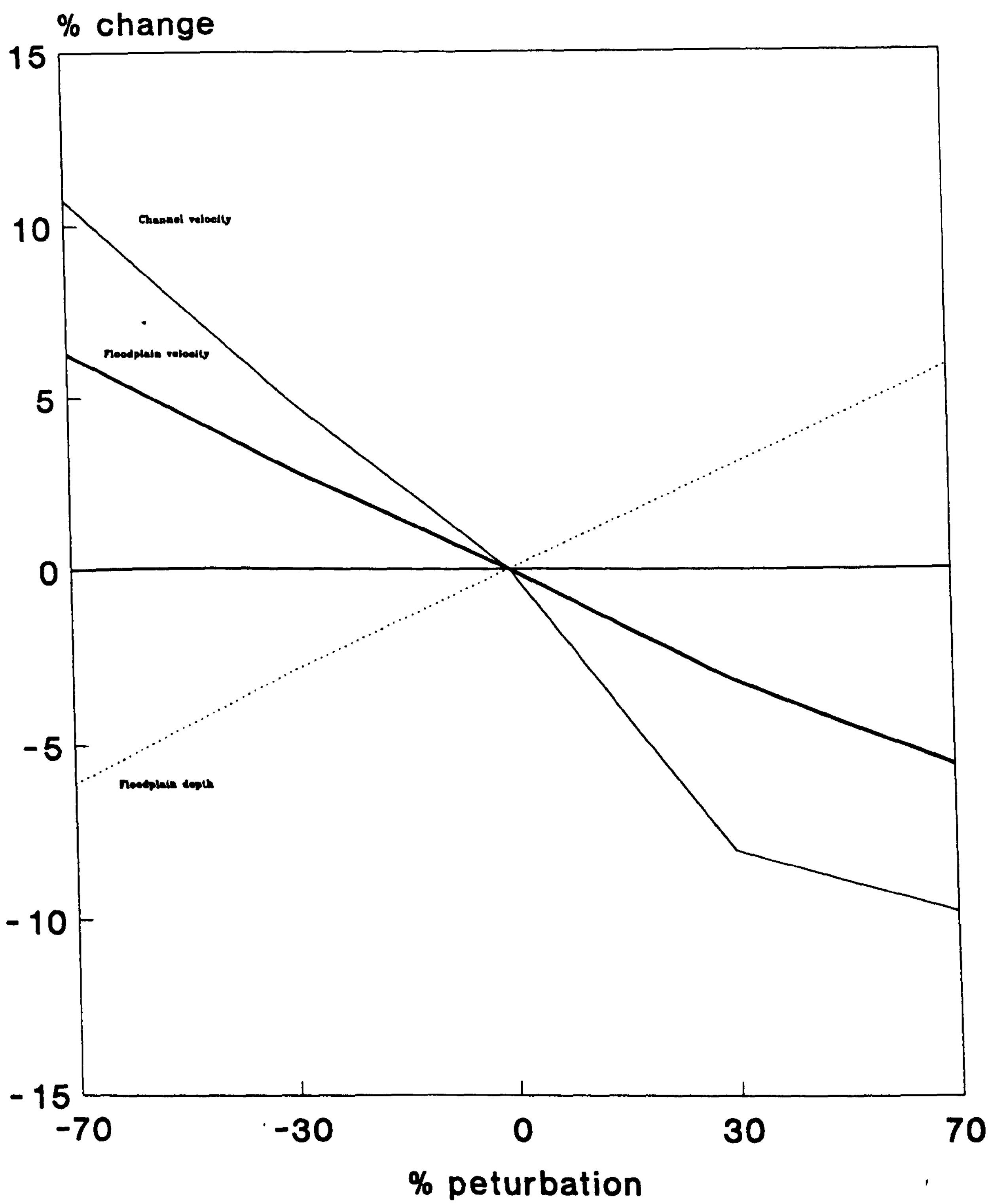


Figure 4.6: Summary of sensitivity analysis results for domain coefficient depth range.

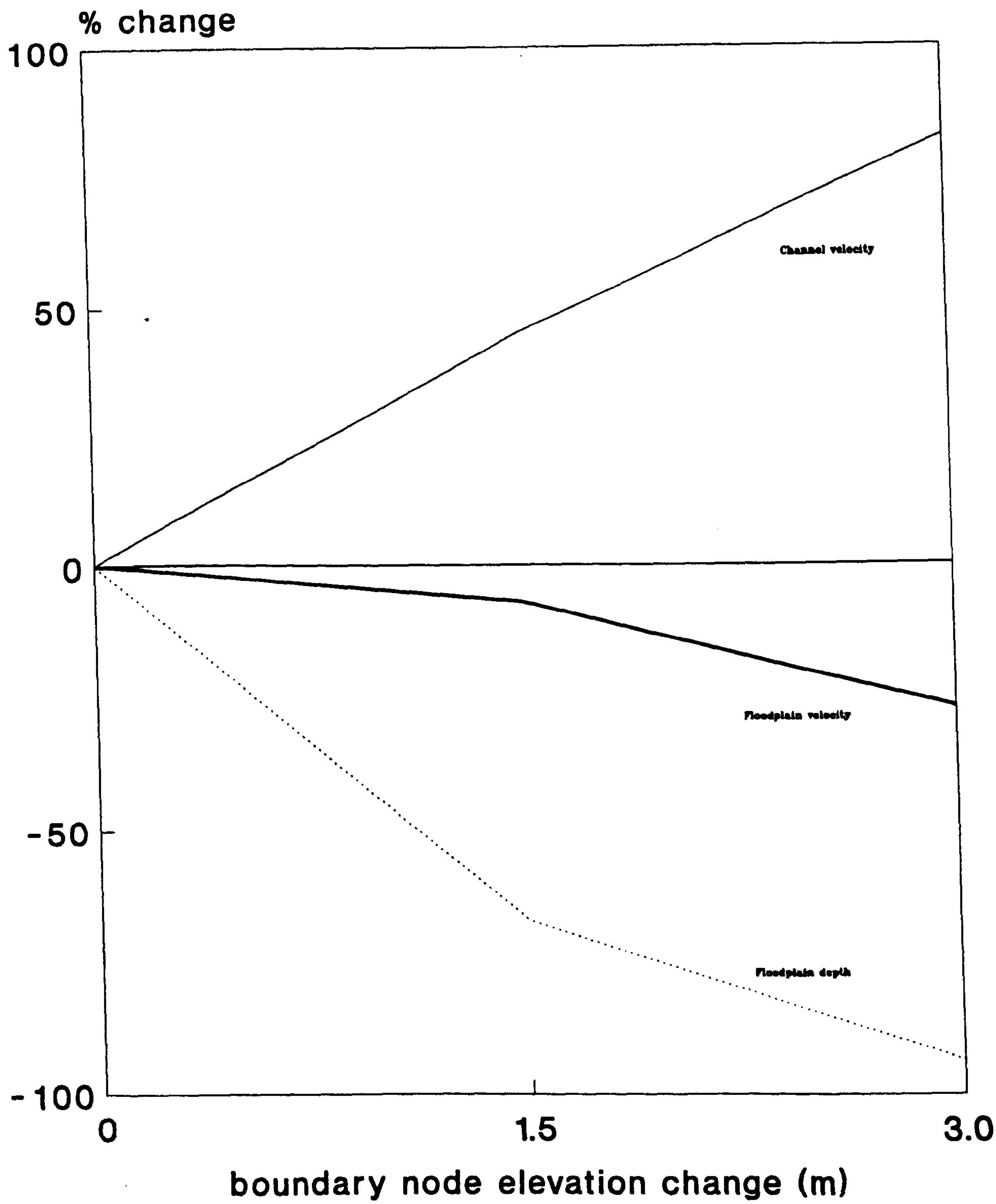


Figure 4.7: Summary of sensitivity analysis results for mesh lateral slope.

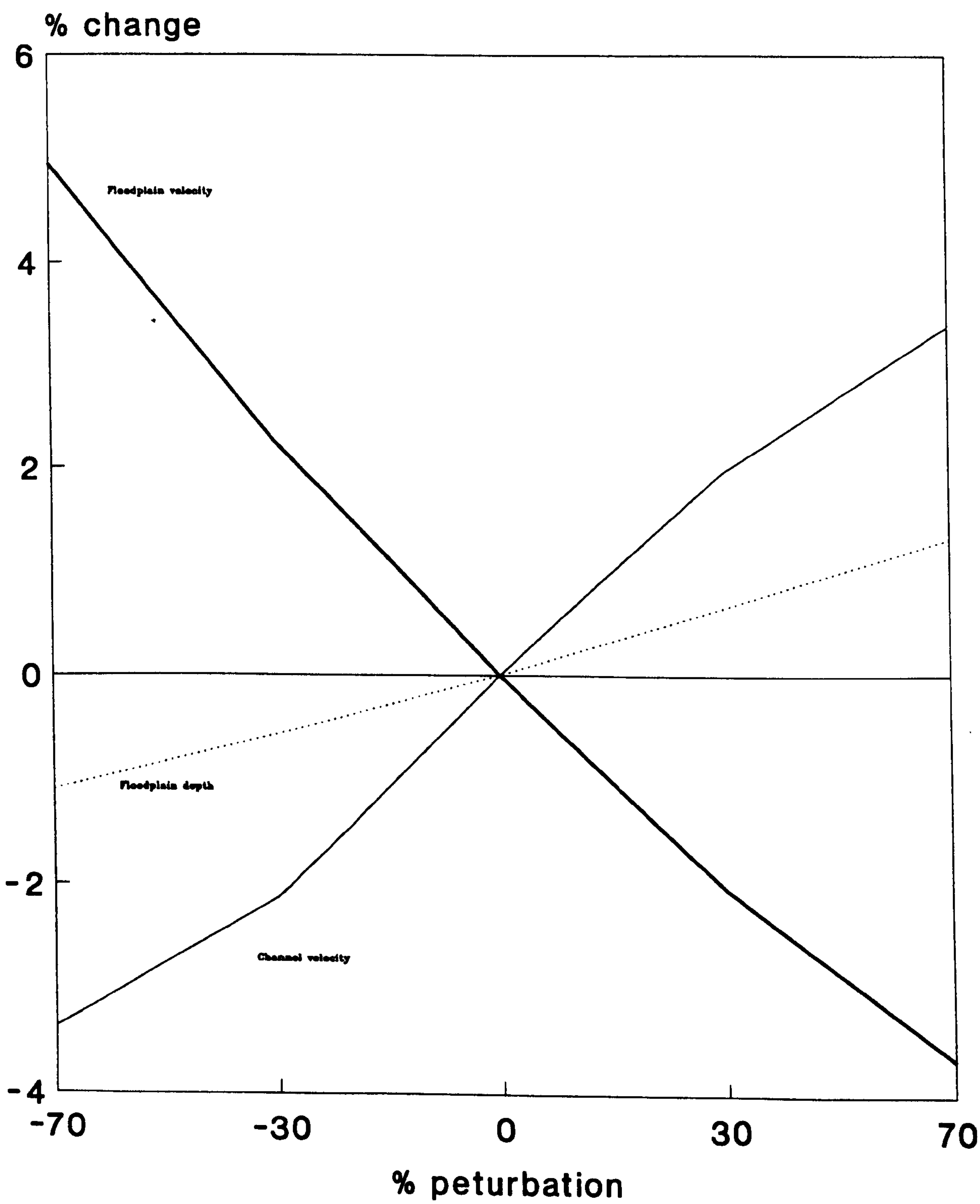


Figure 4.8: Summary of sensitivity analysis results for the boundary friction coefficient.

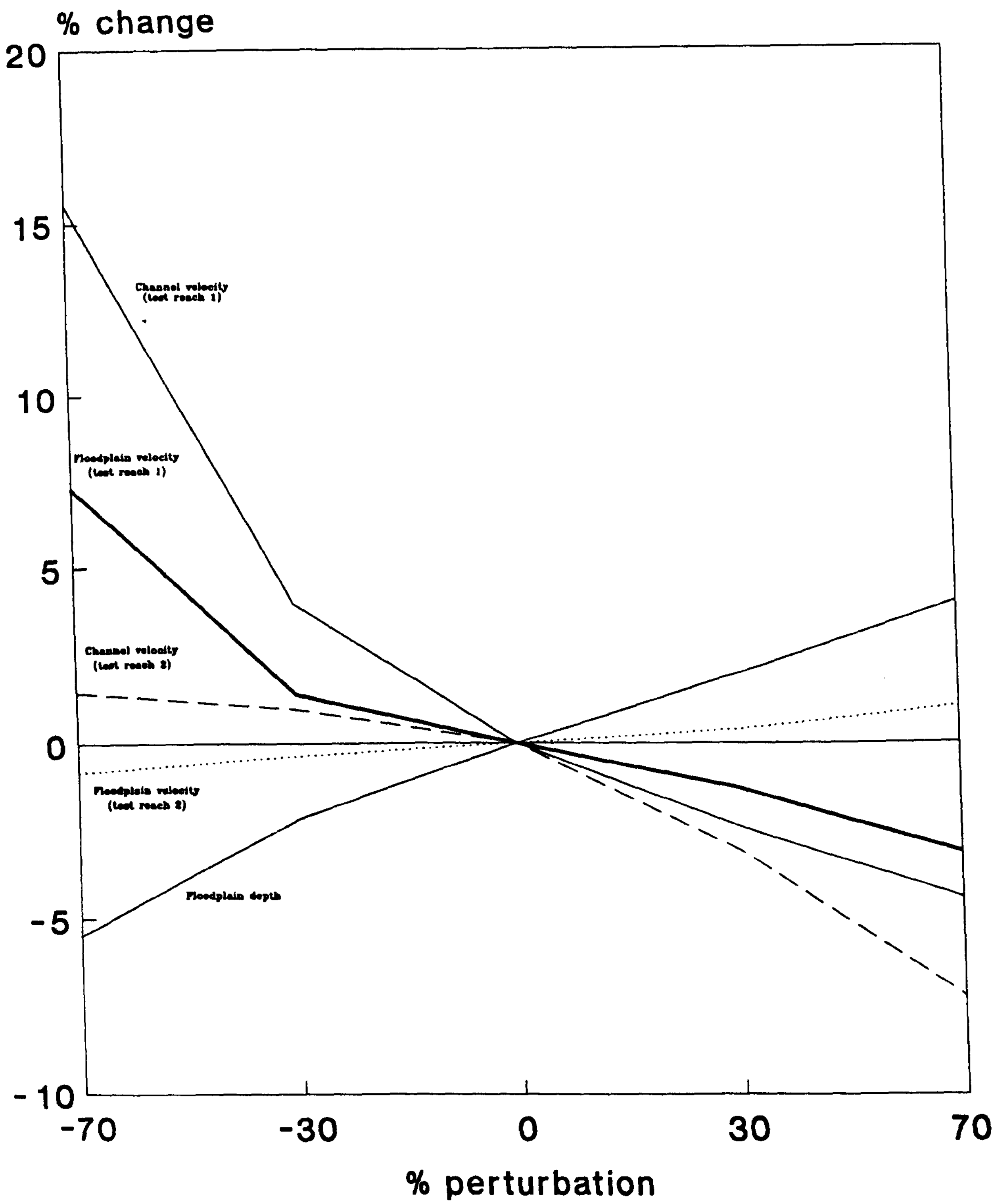


Figure 4.9: Summary of sensitivity analysis results for the eddy viscosity coefficient.

where US_f = unit sensitivity to the factor f,
F_o = model output,
F_i = model input parameter,
n = number of values.

This gives the percentage change in a particular dependent variable caused by a 100% change in that parameter. This is a development of the general definition of sensitivity given by McCuen (1973, 1976) as:

$$S_f = \frac{\partial F_o}{\partial F_i} \tag{4.2}$$

where S_f = sensitivity function of the factor f.

The unit sensitivity of a particular parameter is therefore given in the more usable form of a magnitude rather than a ratio. This also presents the possibility of considering systems of equations where there is more than one dependent variable, each with specific sensitivities to particular parameters. In this case we have considered both velocity and depth independently, noting significant differences in sensitivity. For channel shape and channel sinuosity, where the degree of perturbation cannot be quantified, equation 4.1 reduces to:

$$US_f = \frac{\Sigma\,(\Delta F_o)}{n} \quad x100 \tag{4.3}$$

The result of applying these formulae to the sensitivity analysis results is given in Table 4.1.

Parameter	Unit Sensitivity - velocity	Unit Sensitivity - depth
Eddy viscosity coefficient	7.9	0.87
Boundary friction coefficient	13.38	2.5
Domain coefficient depth range	23.4	12.46
Minimum domain coefficient	0.88	0.012
Time step decrease	1.8	0.68
Longitudinal slope	25.2	25.6
Lateral slope	8.67	8.7
Channel shape	1.65	0.04
Channel sinuosity	2.56	0.08
Mesh resolution increase	5.02	0.016
Rating curve	60.6	50.4

Table 4.1: RMA-2 unit sensitivities for velocity and depth based on a single factor perturbation sensitivity analysis for three flood events and two test reaches.

This allows model response to parameter variation to be ranked via a relative sensitivity index, defined as:

$$RSI = US_{velocity} + US_{depth} \tag{4.4}$$

where RSI = Relative Sensitivity Index.

The results of this ranking exercise are given in Table 4.2.

Rank	Parameter	Relative Sensitivity Index
1	Rating curve	111.0
2	Longitudinal slope	50.8
3	Domain coefficient depth range	35.86
4	Lateral slope	17.37
5	Boundary friction coefficient	15.88
6	Eddy viscosity coefficient	8.77
7	Mesh resolution increase	5.036
8	Channel sinuosity	2.64
9	Time step decrease	2.48
10	Channel shape	1.69
11	Minimum domain coefficient	0.446

Table 4.2: Model parameters ranked in order of sensitivity of response to perturbation.

These two indices provide an effective summary of model behaviour.

4.2.1.3 Model response to parameter variability

Some general remarks concerning the sensitivity analysis data set will be made before proceeding to comment on the results for each individual parameter.

Percentage change in the observed variables tended to decrease with increasing event magnitude. This appears to be due to the fact that a given increment in a particular factor tends to produce the same absolute magnitude change in the dependent variables regardless of event size. Thus, for high recurrence interval events the reference simulation values of peak velocity and depth will be large relative to the impact of particular parameter variation. For this reason also, the percentage change in peak stage is greater for floodplain compared to channel nodes.

The model responds to factor perturbation in a smooth, quasi-linear fashion for all but one of the parameters considered here. This is consistent with the simulation of gradually varied flow with non-linear equations. A perfectly smooth response is not shown by these results due to the complicating dynamic effects introduced by unsteady simulations. We may therefore conclude that sensitivity analysis results are consistent with the mathematical model specified in Chapter 3 and that **this model shows an absence of irregularities for the range of parameters used in its normal operation.**

On the basis of these results the model appears sufficiently sensitive to represent actual flow field variations in the physical system. Although sensitivity in terms of stage may appear low, a small change in stage translates into a relatively large increase in cross sectional area when flow is in an out-of-bank condition. Abnormal sensitivities to particular parameters are not shown and the rank order of parameter sensitivities is in line with expectations.

The ranking of model parameters in terms of sensitivity shows model behaviour to be predominantly conditioned by the downstream boundary condition, topographic data used in mesh construction and the domain coefficient depth range, boundary friction and eddy viscosity parameters. As expected, the model is most sensitive to the stage discharge rating curve used as the downstream boundary condition for the finite element solution. As it is also reasonable to assume that the inflow rate boundary condition at the upstream extremity of the mesh will also be important, we can conclude that flow data used to configure the model largely conditions the accuracy of the solution. This sensitivity data will be compared to parameter estimation errors in Section 4.2.1.4.

The time to peak of both velocity and depth did not show significant variation. Some minor variations do occur, notably for boundary friction and the domain coefficient depth range, however no consistent pattern emerges. This may be due in part to the short scale test reaches used in the sensitivity analysis, which may be too short to allow attenuation effects to be manifested.

The following comments may be made concerning the behaviour of each individual parameter.

i) Eddy viscosity coefficient - mathematically this parameter represents the turbulence structure of the flow. The higher the value of eddy viscosity, the more turbulent the flow. One would therefore expect that increasing the eddy viscosity coefficient would reduce total velocity, as more vigorous chaotic secondary motions in the flow should increasingly dissipate turbulent energy. Correspondingly, stage would be increased to maintain mass continuity. This pattern is demonstrated by the sensitivity analysis, albeit in a more complex fashion. For floodplain nodes in test reach 2 the pattern of change captured by the sensitivity analysis is the opposite of what one would expect from purely theoretical considerations. This is easily explained as the wider floodplain in this test section leads to reduced floodplain depths for a given event. This appears to be precluding the development of fully turbulent flow and therefore reducing the impact of altering the turbulent eddy viscosity coefficient. This is consistent with the mathematical model, although insufficient to fully explain the increase in velocity observed here. To do this the dynamics of the momentum transfer mechanism must be considered. As eddy viscosity acts in this case to reduce channel but not floodplain velocity, the velocity differential between main channel and floodplain flows is increased. This increases the strength of the momentum transfer from the main channel to the floodplain. This additional transfer of momentum to the floodplain is thus sufficient to account for the increase in predicted floodplain velocity. We have therefore verified that the eddy viscosity coefficient is behaving in a manner consistent with the mathematical model. We have also been able to demonstrate the complexity of flow effects that the refined RMA-2 model is capable of simulating. The model's ability to capture aspects of the momentum transfer mechanism, identified in Chapter 2 as being of critical importance for the accurate simulation of compound open channel flows, has also been shown.

ii) Boundary friction coefficient - the sensitivity of the boundary friction coefficient was found to be entirely consistent with expectations. Increasing friction values significantly reduced velocities and increased stage. This confirms the findings of Baird and Anderson (1990) who demonstrated that boundary friction is a useful calibration parameter for RMA-2 applications.

The impact of spatially varied changes in boundary friction was not examined, as this has already been covered elsewhere (Baird and Anderson, 1990).

iii) Domain coefficient depth range - this parameter represents the sub-grid bathymetry in the region of a particular node that may cause it to be either wet or dry. Reducing the domain coefficient depth range implies a more simple floodplain microtopography. Less water will therefore be stored on a particular element during an inundation cycle, with more water running off. A reduction in the value of this parameter will consequently lead to a reduction in depth and an increase in velocity. This is borne out by the sensitivity analysis results. Here again the momentum transfer mechanism operates to create changes in channel velocity even though it is floodplain elements that are undergoing wetting and drying processes. The sensitivity of the model to this parameter confirms the conclusion of King and Roig (1988) that the domain coefficient depth range is another significant calibration parameter, although no guidelines currently exist to relate parameter values to physical reality.

iv) Minimum domain coefficient - this factor controls the small positive depth of water maintained over a dewatering element for the time it remains part of the numerical solution. This is used to overcome computational difficulties with the domain coefficient approach (see Section 3.2.1.2). It is assumed that this water volume has a negligible impact on the solution. Although this was theoretically validated in Section 4.1.1.2 the sensitivity analysis reported here confirms this finding. It was therefore concluded that the minimum domain coefficient has a negligible effect on model predictions for river channel/floodplain simulations at a long reach scale.

v) Time step - A 0.5 hour time step was chosen for RMA-2 simulations as this represented an acceptable balance between data availability (0.25 hour intervals) and the need to ensure acceptable computing requirements. It is, however, important to ascertain if a smaller time step, and therefore increasing precision, has a significant effect on results. The sensitivity analysis shows that increasing the time step from the chosen 0.5 hour basis results in a significant decrease in precision, however decreasing the time step brings little gain relative to the deterioration in computational efficiency. The selection of a 0.5 hour time base for RMA-2 simulations is therefore fully justified.

The following mesh construction parameters were analysed in terms of test reach 1 only.

vi) Longitudinal slope - sensitivity of model output to changes in longitudinal slope is both highly significant and straightforward in operation. Flow velocity and depth show approximately a ±20% change in their peak values for a 50% change in gradient. This is entirely consistent with expectations, but does however imply that the topographic data used to establish the longitudinal slope needs to be sufficiently accurate to meet particular modelling objectives specified by the user. This point will be returned to when the construction of finite element networks for real applications is discussed in Section 5.2.1.

vii) Lateral slope - changes to lateral floodplain slope appear to be a less significant control on model predictions than longitudinal slope. This is not surprising as mass and energy transfers in open channel systems largely occur along the longitudinal axis. Increasing lateral floodplain slope substantially increases channel velocities as the channel has to accommodate excess water previously resident on the floodplain. However, for reasons which are not entirely clear, channel depths do not increase. Those nodes at the outside edge of the floodplain are raised above the water surface elevation and are consequently 'dry'. A small water depth is maintained at these nodes as a consequence of the minimum domain coefficient. Thus the value of the minimum domain coefficient used in RMA-2 floodplain applications results in an actual water depth of approximately 0.09 m being maintained at these 'dry' nodes. This provides further evidence that the volume of water involved in the assumption of a small positive flow depth at dry nodes within the solution is negligible.

viii) Channel shape - three channel geometries were analysed; a prismatic trapezoid (Figure 4.10a), the modified trapezoid described in Section 3.3.2 (Figure 4.10b) and a channel cross section configured with a greater degree of circularity (Figure 4.10c). These modifications from a true prismatic trapezoid were undertaken to improve the simulated flow distribution across the channel. Sensitivity analysis results show that such changes have only a marginal impact, with peak depths almost entirely unaffected. However, it is

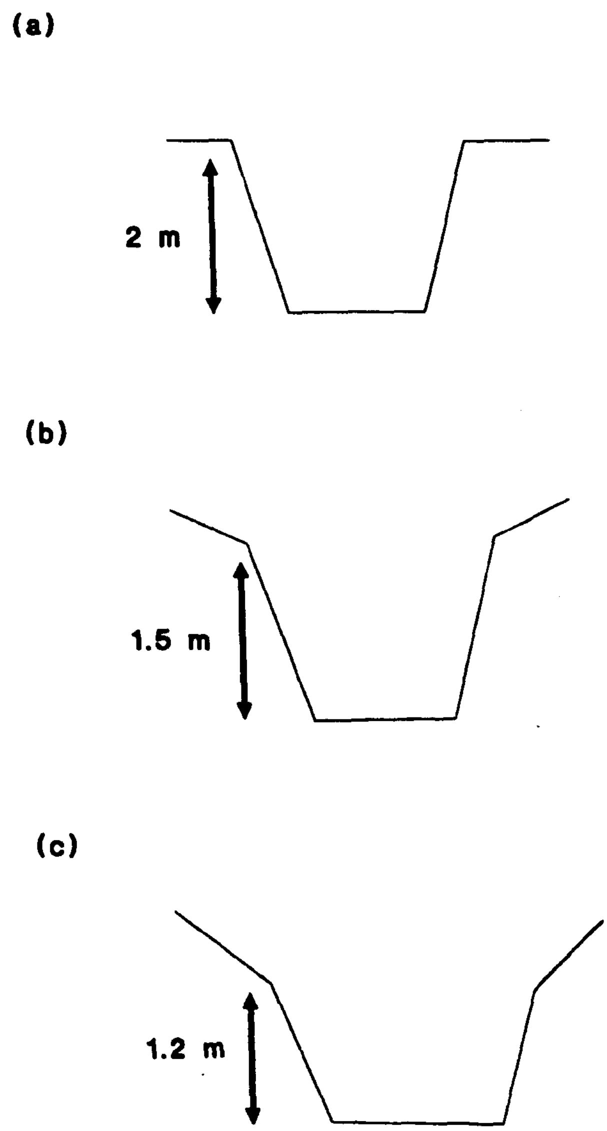


Figure 4.10: Channel cross sections used in the RMA-2 sensitivity analysis. a) Prismatic trapezoid, b) non-prismatic trapezoid developed for use in actual RMA-2 applications, c) a channel cross section configured with a greater degree of circularity.

quite possible that such changes will act to affect the stability of the full solution and further, that the subtle effects relating to channel flow are being masked by the extensive inundation caused by even the smallest event used in the sensitivity analysis (a minimum 0.7 m above bankfull).

ix) Channel sinuosity - three channel sinuosities were tested (see Figure 4.11) to determine the effect of various channel representations. However, the analysis shows only a small effect on predicted velocities. Increasing sinuosity decreases channel and floodplain velocities as the flow is routed through a longer longitudinal distance. Predicted depth does not seem to be affected. We may have expected that channel sinuosity would have a larger impact on model results. Again, this may be due to magnitude of the flood events being simulated, with extensive out-of-bank flow masking the impact of channel-based changes on model simulations. Despite these reservations the results imply that for its intended purpose as a flood event simulator, RMA-2 does not require a highly detailed representation of channel planform for accurate simulation of bulk flow characteristics.

x) Mesh resolution - this aspect of sensitivity testing refers to the effect of altering spatial, rather than topographic, mesh resolution. Two further versions of test reach 1 were constructed in addition to the original, with increased and decreased resolution (see Figure 4.12). Finite element theory suggests that solutions tend to exactness as the element length tends to zero. We therefore wish to ascertain if the resolution of elements used in the proposed long reach applications is sufficiently exact to represent gradients of system variables. Results show that peak depths are not affected by altering the mesh resolution, however velocity, in particular its lateral distribution, is. This can be explained by considering the velocity gradients being simulated by the model. In the longitudinal axis the velocity gradient between adjacent nodes is shallow and the spatial resolution of the mesh is sufficient to model this. Consequently, increasing mesh resolution has no significant effect on the simulated downstream propagation of the floodwave. When considering the lateral velocity gradient, we observe a large velocity differential between the main channel and near stationary water at the outer edge of the floodplain. This gradient is more accurately represented with a grid with a higher lateral resolution. The modeller must therefore exercise caution in selecting an appropriate lateral grid resolution to simulate such lateral velocity

a)

b)

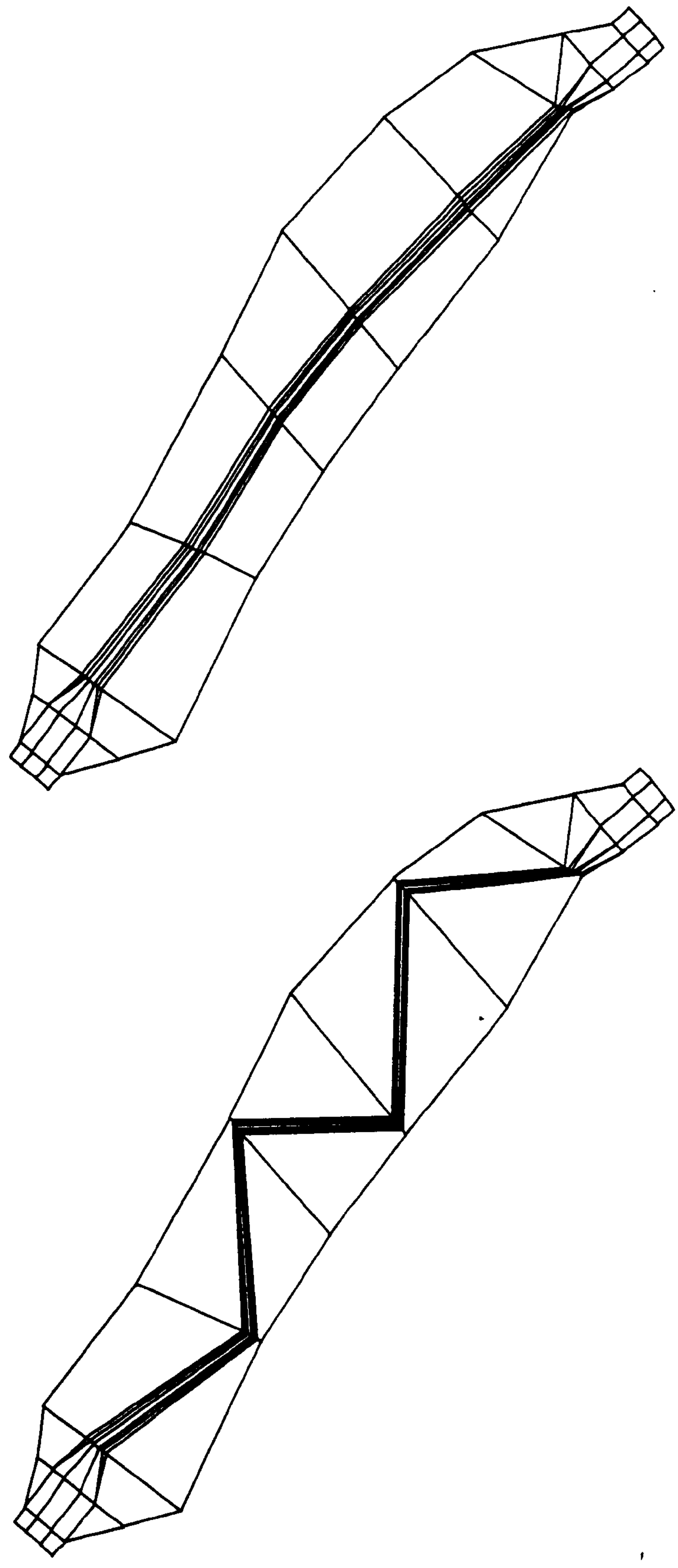


Figure 4.11: Straight and sinuous experimental test reaches used in the RMA-2 sensitivity analysis. a) Straight, b) sinuous.

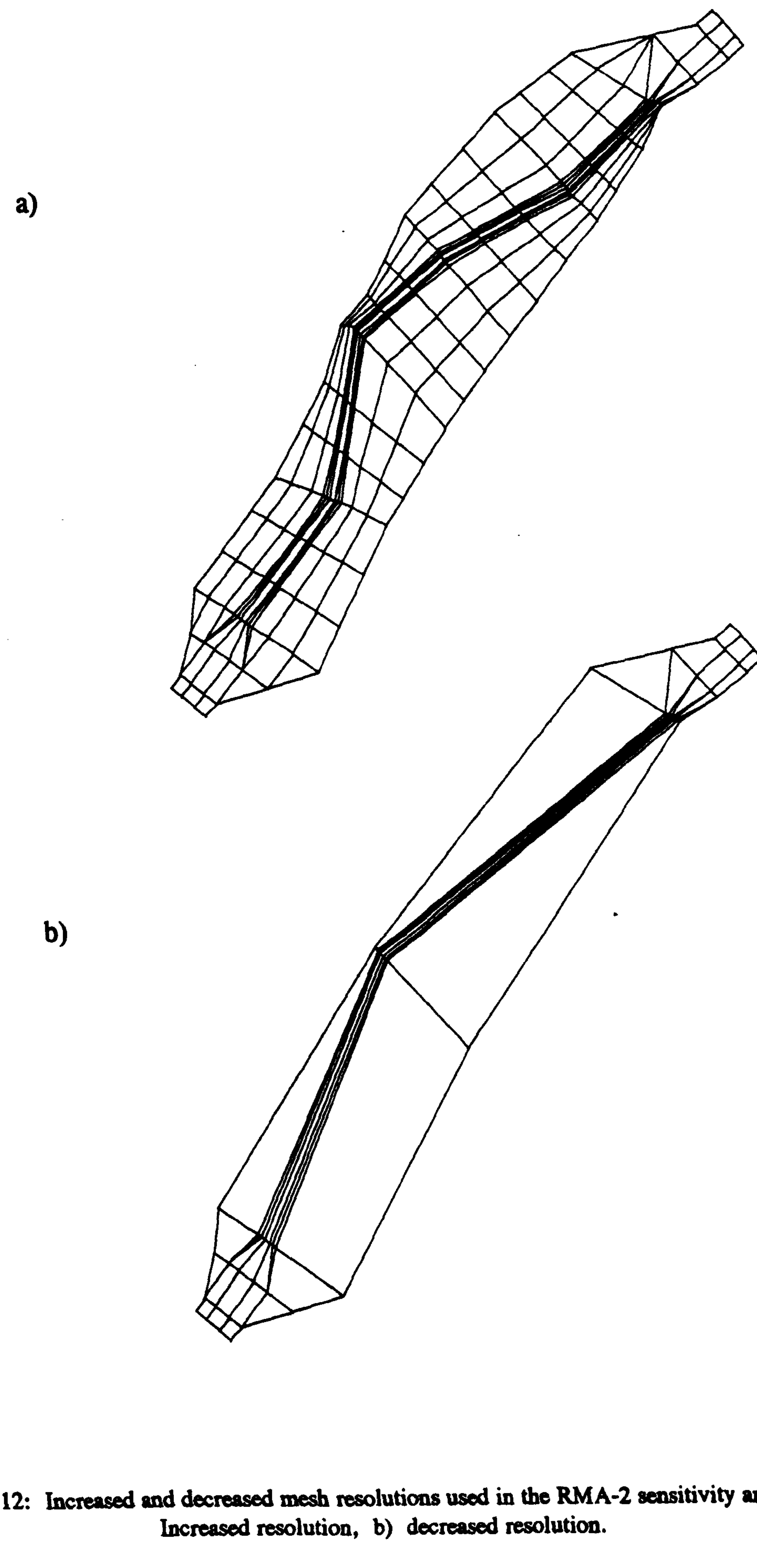


Figure 4.12: Increased and decreased mesh resolutions used in the RMA-2 sensitivity analysis. a) Increased resolution, b) decreased resolution.

distributions at a level of accuracy sufficient to meet application specific objectives. These results indicate that the lateral resolution suggested in Section 3.3.2 for floodplain elements at long reach scales is sufficient to simulate the broad pattern of lateral flow velocity and achieve the objective, outlined in Chapter 2, of simulating flow variables at horizontal scales of 10 - 100 m. For applications requiring more detailed predictions, this analysis shows that lateral mesh resolution should be increased.

xi) Stage discharge rating curve - this forms the downstream boundary condition for the finite element scheme. The parameters of this relationship were manipulated to give a percentage change in the discharge predicted for a given stage. Unsurprisingly, the model is highly sensitive to changes in this parameter. This is of particular importance in the light of the potentially large errors involved in the estimation of stage discharge rating curves for compound channels.

4.2.1.4 Parameter sensitivity and field estimation error

Unit sensitivities derived from the analysis were used in conjunction with the typical parameter estimation errors given in Table 3.1 to give maximum potential errors in velocity or depth for each parameter. This allowed the potential uncertainty in model predictions caused by these errors to be quantified. This data is given in Table 4.3.

Two problems exist with the error estimates given in Table 4.3. Firstly, the estimates of parameter error are only typically expected values and will obviously not hold for all situations. Secondly, the values derived for unit sensitivity are based on a analysis which can never be fully comprehensive. We can however be certain that the error estimates are of the correct order, even if they are not precisely correct. Table 4.3 therefore gives a good indication of the typical errors associated with each parameter of the refined RMA-2 model for floodplain applications. In particular, the rating curve estimation error is highlighted as being of critical importance to the overall accuracy of the refined scheme. A suitably accurate method of rating curve estimation in compound channels should therefore be selected. However, maximum potential errors compare favourably with the accuracy of ±8% associated with two dimensional depth averaged model codes applied under ideal conditions (Norton *et al.*, 1973; Hervout, 1989). It has also been demonstrated that although a degree of error will be introduced by the topographic discretization, these are small when compared to rating curve induced errors. **Accuracy of the scheme is therefore predominantly driven by the accuracy of the flow data used to set up the finite element boundary conditions for each dynamic simulation.**

Parameter	Typical estimation error (±%)	Potential velocity error (±%)	Potential stage error (%)
Rating curve - single/divided channel - HYMO3 - rated section - HR design manual	 25 15 5 3	 15.2 9.1 3.03 1.82	 12.6 7.56 2.52 1.51
Longitudinal slope	10	2.52	2.56
Domain coefficient depth range	10	2.31	1.25
Lateral slope	10	0.87	0.87
Boundary friction coefficient	3.3	0.44	0.08
Eddy viscosity coefficient	20	1.58	0.174
Mesh resolution increase	User specified	-	-
Channel sinuosity	5	0.13	0.004
Time step decrease	User specified	-	-
Channel shape	5	0.08	0.002
Minimum domain coefficient	3	0.02	3.6×10^4
Range of maximum potential error for various rating curve estimation methods		±9.77 - ±23.15	±6.45 - ±17.54

Table 4.3: Maximum potential error in model dependent variables caused by typical parameter estimation errors.

4.2.1.5 Wider issues of sensitivity analyses for complex environmental models

Section 4.2.1 has demonstrated the difficulty of performing meaningful sensitivity analyses for complex environmental models such as RMA-2. It appears questionable whether a complete sensitivity analysis for such models could ever be undertaken given the almost infinite range of flow and parameter conditions that would require combinatorial stochastic testing and the highly distributed prediction fields that would result. Sensitivity analysis is therefore only able to sample a subset of possible behaviours, which are presumed to represent total model performance. Delimiting this subset therefore requires a number of simplifying assumptions to be made concerning which parameterizations are representative of this larger population of possible behaviours. At present, no proscriptive method exists for carrying out such an analysis, with simplifying assumptions and decisions concerning the effectiveness of particular research designs currently made on an *ad hoc* basis.

This thesis has adopted the view that the most appropriate sensitivity analysis research design is one that provides all required information at minimal computational cost. This approach is adequate in the absence of any other guiding criteria, but is ultimately unsatisfactory as it does not ensure representative sampling of model behaviours. As environmental models become more complex, proscriptive methods for their evaluation, based perhaps on sampling theory, will become increasingly necessary.

4.2.2 Numerical stability of the refined RMA-2 scheme

Stability of iterative numerical methods can be defined by the continuity and convergence of the solution. Maintenance of continuity ensures that mass is being neither gained nor lost by the model. As this cannot be ensured theoretically (see Section 3.2.1.1), continuity statistics for the refined RMA-2 model are calculated on the basis of the flow crossing a line that transects the entire region of fluid movement perpendicular to the main flow axis for a steady state solution. Flow at such lines should be identical for all points on the network if there are no mass continuity errors. In reality, however, variations in the continuity statistic will occur as it is impossible to select a line perfectly orthogonal to the flow axis. Consequently, a solution where mass continuity was maintained, and hence was stable, was defined as one where the flow at such transect lines was within ±2% of that entering the reach. This figure has been suggested by Norton *et al.* (1973) as being indicative of adequate model performance for this class of scheme. Continuity values for a typical steady state simulation are given in Table 4.4. A number of such steady state simulations were examined and the continuity found to be always within the above limit. The numerical scheme used in the refined RMA-2 model therefore maintains continuity for river channel/floodplain applications at a long reach scale.

The stability of the iterative procedure is defined by the rate of convergence of successive approximations. For unstable solutions the iterative numerical procedure fails to converge on a single value, giving instead greater estimates of the dependent variables at each iterative step. The convergence statistic, given as percentage change in dependent variables between iterations, will thus tend away from zero exponentially. Convergence statistics in the range ±0.0001 - 0.2% at the end of an iteration cycle were classified as stable. The sensitivity analysis demonstrated that such stable solutions could be achieved for floodplain applications, and that this occurred within 4 - 6 iterations for each time step. We can therefore conclude that the refined RMA-2 scheme is numerically stable for river channel/floodplain applications at the scale proposed in this study. This issue will be returned to when operational stability limits for real applications are discussed in Section 5.2.4.

Line number	Continuity (%)
Inflow	100.0
Outflow	100.0
1	100.9
2	98.3
3	100.3
4	99.9
5	100.5
6	99.8
7	100.0
8	99.1
9	99.9
10	101.9
11	101.0
12	101.3

Table 4.4: Continuity values for a typical RMA-2 steady state simulation.

4.2.3 Model response to abnormal parameterization

The final facet of computerized model verification is the exposure of the model to parameter values outside the range of those normally encountered. This allows the user to test for areas of model breakdown and irregularity which may indicate potential problems with the computer code. Three aspects of the model were tested with abnormal parameter values for both steady state and dynamic simulations:

i) the group of parameters identified by the sensitivity analysis as having the largest influence on model output, namely, the downstream boundary condition, the topographic survey, the domain coefficient depth range and the boundary friction coefficient,

ii) the effect of reducing the minimum domain coefficient,

iii) model response to low flow conditions.

Examination of the most sensitive parameters is a logical area to test for model breakdown as changes in these have proportionally the largest effect on the solution. This demonstrates the utility of a first pass sensitivity analysis in isolating areas for further investigation.

Reduction of the minimum value of the domain coefficient allowed identification of the smallest possible value of this parameter that still allowed stability to be maintained for long reach scale applications. Identification of this parameter value allowed simulations to be constructed where the volumetric error associated with the assumption of a small positive flow depth at 'dry' nodes was minimized.

Low flows were thought to be an area of potential model breakdown, given that the refined RMA-2 model was specifically designed as a flood event simulator. In addition, it was also important to assess the ability of the wetting and drying routine to cope with the large change in the number of elements in the solution occurring at inflow rates below bankfull discharge.

The effects of imposing abnormal values of these parameters are given in table 4.5.

Aspect/parameter	Stability range
Stage discharge rating curve	-90% - 120% error in the prediction of discharge for a given stage compared to the original curve.
Topographic representation	Some instability caused for highly complex topographic representations. Discussed in detail in Section 5.2.4 in the context of specific applications
Domain coefficient depth range	0.3 m - > 2.5 m
Boundary friction coefficient	0.02 - > 0.5.
Minimum domain coefficient	Model unstable at < 0.015
Low flows (bankfull = 26 m^3s^{-1})	Steady state simulations: unstable at inflow rates < 24 m^3s^{-1} Dynamic simulations: unstable at inflow rates < 17 m^3s^{-1}

Table 4.5: Summary of model response to abnormal parameter values

These tests therefore give the limits to system operation for long reach floodplain applications. From this we can state that the areas of system breakdown discussed above will not constrain applications of the refined RMA-2 model. This procedure has also allowed a stable minimum value for the domain coefficient to be established. Finally, we can conclude that RMA-2 is only suitable as a flood event simulator, dealing with flows from bankfull discharge upwards. This is sufficient to allow the scheme to fulfil the modelling objectives stated in Chapter 2. This instability at low flows is linked to an apparent inability of the wetting and drying routine to cope with large numbers of floodplain elements of considerable areal extent dropping out of the solution at inflow rates below bankfull discharge. It is suggested that this situation may be improved by the use of smaller floodplain elements if the simulation of low flows were necessary.

4.2.4 Summary of computerized model verification

Section 4.2 has detailed the first comprehensive computerized model verification of the RMA-2 scheme. In particular the refined model's suitability for application to floodplain environments has been theoretically confirmed. In such situations the model has been shown to be stable over typical parameters ranges, behaving in a manner consistent with the physical system and the mathematical model. **The refined RMA-2 computer model has therefore been verified for problems concerning the simulation of river channel/floodplain flow at long reach lengths.**

CHAPTER 5

Model implementation and initial validation of simulated process behaviour

Having undertaken an initial evaluation of the refined RMA-2 scheme in Chapter 4, it is now possible, following the research design outlined in Chapter 3, to proceed with application of this theoretically validated model to a real study reach. This procedure will allow operational rules for such applications to be established and provide confirmation of the conclusions drawn in Chapter 4. In addition, an initial comparison of model predictions with observed data for the downstream system outlet will be facilitated and issues of parameterization and data availability will be highlighted in the context of a real application. Further validation of simulated process behaviour will be undertaken in Chapter 6, with an evaluation of model predicted flow field information for the entire reach. In particular, the model's ability to simulate dynamic inundation processes at a high level of spatial and temporal resolution will be assessed.

5.1 Selection of a study reach

As indicated in Section 3.4 a single study reach was selected for application of the refined RMA-2 scheme due to the time constraints involved in constructing, validating and developing operational rules for a previously untested finite element model. It is however acknowledged that while conclusions drawn will be appropriate to other floodplain environments with similar characteristics, the results must be interpreted in this context.

In a previous application of a prototype version of RMA-2 to a floodplain environment (Gee *et al.*, 1990), use was made of a topographically simple study reach for which the quality of available data was high. It is recognised, however, that the majority of floodplain environments do not fall into such a category. The enhancement of the RMA-2 model described in Chapter 3 allow the scope of two dimensional finite element schemes to be extended to consider the more usually encountered data-poor floodplain environments, with an increased range of topographic complexity. It was therefore necessary to select a study reach where these issues could be fully explored and where adequate field verification of the RMA-2 scheme could take place.

An attempt has been made to apply the refined RMA-2 scheme to the lower reaches of the River Culm, Devon, UK. This site fulfilled the above criteria, possessing a number of complex topographic features (mill races, bifurcations, embankments, a sinuous meandering channel and changes in floodplain orientation) and a level of data availability that would allow testing of proposed strategies for dealing with such typically encountered problems. In addition, the regularity of overbank flood events causing substantial floodplain inundation and the presence of a number of detailed long term investigations into floodplain sedimentation (Walling and Bradley, 1989; Walling *et al.*, 1992) were further deciding factors.

5.1.1 Description of the study reach

The River Culm (see Figure 5.1) is an eastern tributary of the River Exe, joining the main river 3 km north of Exeter. The catchment has a total area of 276 km^2; with the reach of interest being the 11 km section of floodplain between Cullompton and Stoke Cannon. In its lower reaches the Culm meanders across a well developed floodplain, averaging about 450 m in width. Along much of this reach the river flows in a gravel-bed channel approximately 12 m wide, with bankside levées of fine alluvial material up to 1 m in height. Overbank flooding is relatively frequent in the winter months with substantial inundation typically occurring on six occasions a year (Bates *et al.*, 1992). During a major flood approximately 5.5 km^2 of floodplain between Cullompton and Stoke Cannon is inundated, with the depth of inundation varying with local topography. Typical inundation depths for the middle of the study reach are 40 cm for the mean annual flood and 70 cm for a 50 year flood. Land use on the floodplain is largely permanent pasture for silage production and cattle grazing, although the latter is confined to the summer months by winter flooding.

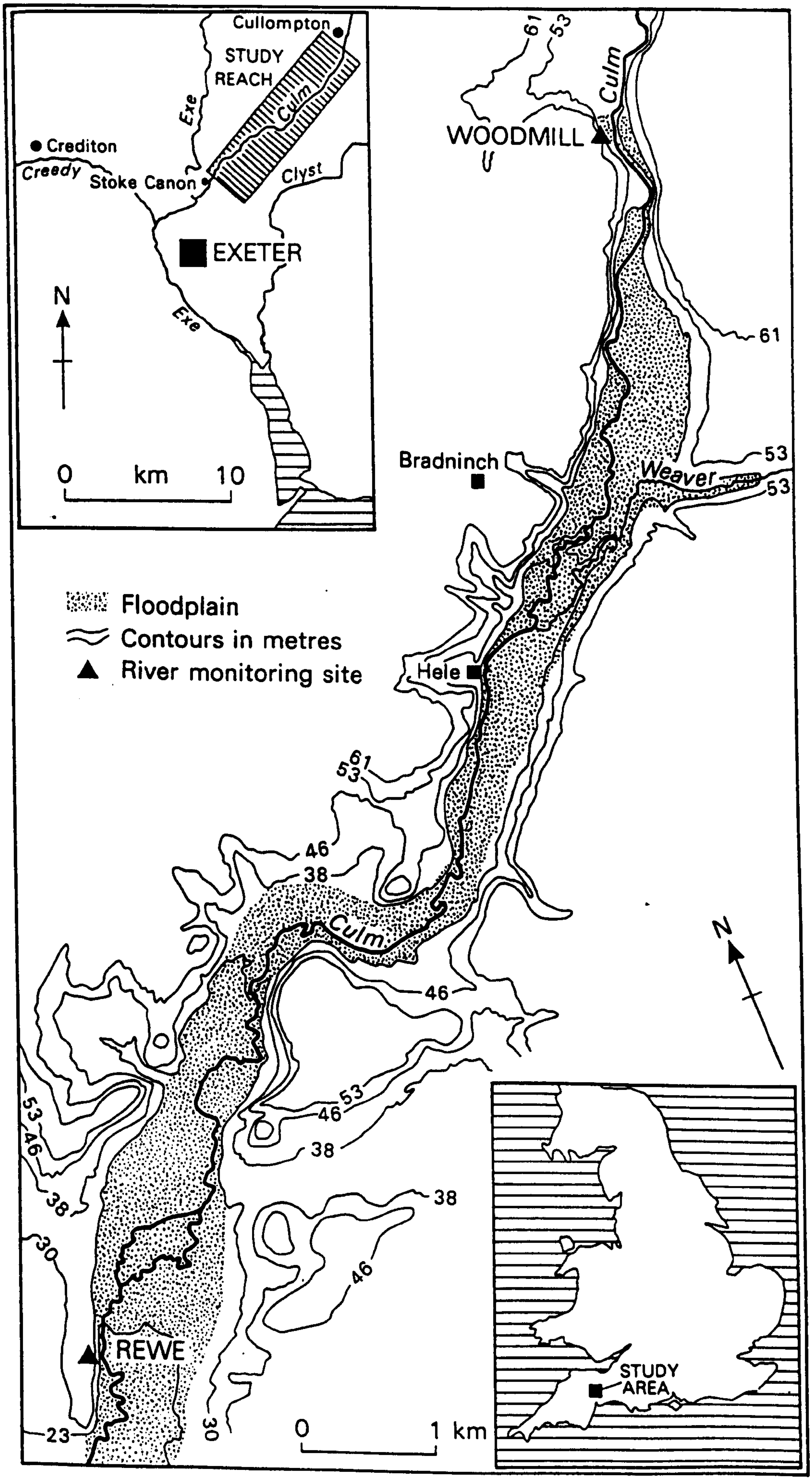


Figure 5.1: The River Culm study reach.

5.1.2 Existing research programmes in the Culm basin

As indicated above, a major research effort is currently under way in the Culm basin to investigate the nature of the sedimentary environment within temperate alluvial floodplain systems. This research effort has developed along two lines of enquiry:

i) an examination of conveyance losses within the whole floodplain reach,

ii) a detailed examination of contemporary rates of floodplain sedimentation.

The former approach has relied on the suspended sediment monitoring programme described in Section 5.2.1 to estimate the amount of sediment entering and leaving the reach for a variety of flood events (Walling *et al.*, 1986). This programme has demonstrated marked differences in conveyance losses between events, varying between 8 and 53% for 15 flood events. This represents an annual deposition of approximately 1750 t between Cullompton and Stoke Cannon, with the majority of this sediment entering floodplain stores.

The latter approach has attempted to confirm this reach length estimate of process rates and in addition has attempted to examine spatial variation in sediment deposition rates at a number of different scales. Point measurements of deposition rates have been made using groups of sediment traps sited on the floodplain. Estimates of average sediment deposition over the last 30 years have also been made by applying caesium^{137} dating techniques to floodplain sediment cores. Such studies have been carried out at reach (Walling *et al.*, 1986), and local (Walling *et al.*, 1992) scales. These results broadly confirm the above estimate of process rates for this basin, demonstrating the extreme spatial variability of floodplain sediment deposition and its relationship to floodplain topography.

5.2 Configuration of RMA-2 for the River Culm study reach

5.2.1 Available data and model construction

Two gauging stations have been established at Cullompton (upstream) and Rewe (downstream) to provide the flow data necessary to formulate model boundary conditions. Stage recorders installed at these rated sections were used in conjunction with an empirically derived stage-discharge rating curve to estimate discharge at 15 minute intervals. Suspended sediment concentrations are continuously monitored at these points using optical turbidity meters. Sediment loads are dominated by clay- and silt-sized material, with concentrations rarely exceeding 1000 mgl^{-1}. An empirical flood frequency relationship has also been established for the Cullompton gauging station. This flow data, along with the topographic information available from UK Ordnance Survey maps represents the minimum level of data availability required to implement the refined RMA-2 model. Additional relevant data from National Rivers Authority archives includes a map of maximum flood extent for the Culm and a limited number of air and ground photos of large (> 1 in 5 year recurrence interval) flood events.

The following specific data sources were therefore used in the construction of the finite element model:

i) UK Ordnance Survey topographic maps,

1:25OO series no's	ST0205-0305
	ST0005-0105
	ST0004-0104
	ST0003-0103
	ST0002-0102
	SS9802-9902
	SS9801-9901
	SS9601-9701
	SS9600-9700
	SS9400-9500
	SS9499-9599
1:25 000 series no's	SS80/90
	SS81/91
	ST00/10
	ST01/11

ii) NRA map of maximum flood extent,

iii) air photos of flood events; 19/12/83, 27/1/84, 3/4/87,

iv) ground photos of flood events; 15/2/90, 21/12/89,

v) ground photos and field sketches based on site inspection; 30/10/89, 14/11/89,

vi) surveyed cross sections for 7 locations on the study reach, one at each gauging station and 5 at intermediary locations,

vii) stage discharge relationships for each gauging station.

Data sources (i) and (vi) were used to establish the geometry of finite elements forming the mesh and their three dimensional nodal coordinates. The minimum data requirement for this procedure was found to be topographic map coverage at 1:2500 scale and surveyed cross sections at the up- and downstream gauging stations. Intermediate cross sections were used to verify that information derived from Ordnance Survey maps were a reasonable representation of topography at the scale under consideration. This was found to be the case. In areas of insufficient topographic data provision such surveys can be used to supplement this minimum requirement for bathymetric information. Topographic data is also used to establish the domain coefficient depth range parameter used in the RMA-2 wetting/drying routine.

Sources (ii), (iii) and (iv) were used to define system limits. This was characterised as the maximum inundation extent. This area can be termed the 'active' flood. Along much of the reach this area lies between the railway and motorway embankments sited within this particular river corridor. Flow passing beyond these limits into 'inactive' stores of ponded water was treated as a loss to the system and not modelled. This is a reasonable assumption given the small (1 - 2%) volume of such stores compared to the total flood and their lack of direct impact on the flow mechanics of interest. Data sources (ii), (iii) and (iv) were also able to provide further information concerning detailed floodplain topography, and give an indication of those structures identified from the topographic survey that have a significant impact on the flow field. For example, it is obvious from aerial photographs that the railway embankment crossing the floodplain at Hele causes a substantial ponding effect. It was thus necessary to include this feature in some detail in the finite element discretization.

Documentary evidence from site inspections allowed Manning's n values to be derived by applying Chow's (1959) photographic interpretation method to the whole reach. This allowed each element to be assigned to one of a series of ten representative roughness classes. These varied from 0.03 for a clear channel to 0.055

for a wooded floodplain. The roughness classes therefore reflect both the vegetative and hydraulic properties of the reach. Other model parameter values were assigned on the basis of previous successful applications and the guidelines set out by King (1988).

Finally, boundary conditions for the numerical solution were derived from the gauging station data. For the Cullompton gauge a rated section exists. The rating curve relationship provided by this information was therefore used to configure this boundary condition to a high degree of accuracy (±4%). For the Rewe gauge, however, the established rating relationship contains a number of inherent errors which preclude its use in model parameterization and assessment. Strategies for the solution of this problem are discussed in the following Section.

5.2.2 Model parameterization issues

Regarding parameterization for spatially distributed environmental models, Beven (1989) has pointed out the difficulties involved in establishing element averaged values of spatially heterogeneous parameters. In the model it is assumed that a parameter is homogeneous for a particular element. However, known spatial and temporal heterogeneities in the physical system often mean that this 'effective' value represents the sum of a number of other processes. For example, boundary friction has been shown to have a dynamic relationship with flow depth. The value of the this coefficient will therefore vary during the course of a flood event. Despite this RMA-2 assumes a stationary value for this parameter. Beven (1989) notes that no theoretical framework for this lumping of subgrid processes exists and we are consequently left with schemes that can only be applied in conjunction with a programme of field measurements to provide a check on results. Where rigourous model parameterization requirements exceed the quality of normally available data, we are driven to calibration to fully exploit model potential. This is the case with the River Culm application of RMA-2. Previous calibrations for such models have typically involved extensive manipulation of model parameters to ensure satisfactory levels of correspondence between observed and predicted results (see for example Evans and Lany, 1983). For this study an attempt was made to configure the model at a lower level of calibration to avoid possible artificial manipulation of model parameters. Data for calibration has been derived from the sensitivity analysis reported in Chapter 4 and consists of refinement of the boundary friction parameter set only. Specifically

- two values of floodplain boundary friction were tested for each simulation, both taken from this parameter's observed field range. It was therefore possible to undertake an assessment of the effectiveness of such calibration scenarios.

When considering data availability for floodplain environments, sufficiently detailed data may not exist to construct an adequate finite element representation. In the case of RMA-2, the provision of model boundary conditions may be problematic due to the need for a sufficiently long time series of flow data to establish an empirical rating curve. In particular, few such data exist for out-of-bank flows. This can therefore create problems with the assignment of realistic inflows outflows to the finite element continuum for flood events. In terms of the River Culm application, a full compound channel rating curve exists for the Cullompton gauge. However, this is not the case for the Rewe gauging site where the empirical rating curve applied to in-bank flows only and thus needed to be replaced. It should be noted that due to the presence of a control structures at the downstream end of the mesh, the point at which model predictions are extracted for comparison to observed data at Rewe is approximately 100 m upstream of the point at which the boundary condition is applied.

This lack of data for the Rewe gauge allowed the changes to boundary condition determination method proposed in Section 3.3.1 to be assessed (see Section 5.3). This approach uses the HYMO3 modelling package (see Section 2.2.2.1) to derive a stage discharge rating curve from a surveyed cross section and the Manning equation. This flow model has been further developed (Baird and Anderson, 1990) to include discrete multiple routing zones and the effects of momentum exchange between main channel and floodplain flows based on the analytical relationship derived by Knight and Hamed (1984). All data for this method is already available from the information sources outlined in Section 5.2.1. Parameterization of the finite element model was therefore achieved on this basis.

Lack of an adequate rating curve for the Rewe gauging station also leads to problems with conversion of stage to discharge data for validation of simulated process behaviour. The model can be directly validated against stage data, however it is important to investigate all aspects of model performance to fully validate the model. A separate approach to the problem of stage/discharge conversion to that used for model parameterization was taken to ensure independence of observed and predicted results. Initially, the supplied rating curve for this gauging station was linearly extrapolated to allow discharge estimates for out-of-bank flows to be made. This was likely to be problematic, as it has been noted (Section 2.1) that a discontinuity in the stage discharge relationship occurs at bankfull due to the geometry of the compound channel and the development of a shear layer between main channel and floodplain flows (Knight, 1989). This would lead to the linearly extrapolated rating curve underestimating the actual discharge for out-of-bank flows.

Field evidence suggests that such a discontinuity in the rating curve exists at bankfull for this site. The floodplain at Rewe is constrained on the right hand bank, whilst the left hand floodplain is approximately 450 m wide. The impact of exceeding bankfull discharge on the form of the rating curve will thus be significant, with large increases in discharge resulting in only small increases in stage. Verification of the model with the available rating curve was thus liable to be inconclusive and two additional methods of developing the stage data were considered to improve the potential for obtaining maximum information from the data set. Firstly, the in-channel flow component was derived from the available rating curve and compared to model predictions. Secondly, a rating curve estimated for the out-of-bank flow was used to re-assess the stage data. These two methods are detailed below.

Finding an approximate estimate of the in-bank portion of the flow relies on the assumption, borne out by topographic survey, that channel geometry at the Rewe gauging station is basically rectangular (see Figure 5.2). For this channel geometry, discharge is equally proportional to stage over the whole of the channel depth. This also holds true if we extrapolate the channel boundaries vertically. Therefore, in this case, the linear extrapolation of the rating curve provided an estimate of this portion of the flow. For comparison, the channel flow component predicted by RMA-2 was also obtained.

The second method by which model verification was attempted involved estimating the out-of-bank portion of the rating curve at Rewe. This was achieved by estimating the flows associated with floodplain inundation and adding these to the extrapolated rating curve for in-channel discharge. The floodplain flows were estimated from detailed topographic surveys and the Manning equation using the slope-area technique (Gregory and Walling, 1973, p133). The resultant rating curve is likely to be only approximate, as measurement errors in the procedure lead to inaccuracies of the order of ±20% in the discharge estimate for a particular stage, particularly in view of the low gradient of the flow line in floodplain systems. The method by which the rating curve is generated is essentially similar to the HYMO3 approach. However, it is not bound by the constraint of approximating the stage discharge regression relationship to a single function to satisfy model boundary conditions. The final form of the rating curve estimated by the slope-area method (shown along with the original

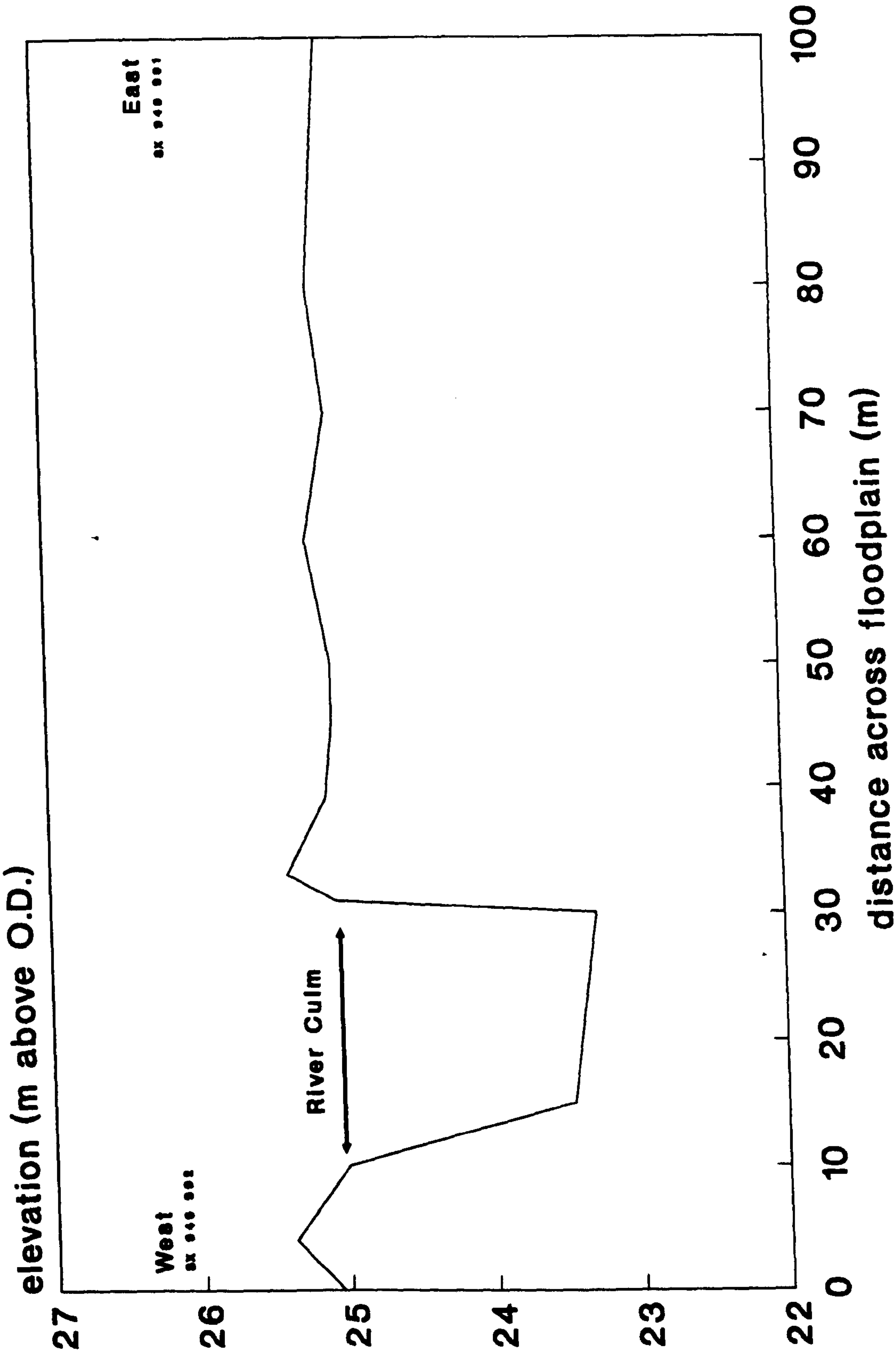


Figure 5.2: Cross sectional geometry for the River Culm at Rewe, showing rectangular channel and bankside levées.

in Figure 5.3) has therefore been approximated to three segments, corresponding to in-channel, bankfull and out-of-bank flow conditions.

It has therefore been demonstrated that even for models, such as RMA-2, with relatively parsimonious parameterization requirements, issues of either unrepresentative or inadequate model parameters are still a significant problem. No generally applicable theoretical solutions to the former problem can currently be identified in the literature (Beven, 1989). A currently unresolvable necessity therefore arises to make what Beven terms a 'conceptual leap' in assuming spatially and temporally averaged model parameters are representative. For the second problem, however, we have identified specific and unique methods of developing available data to overcome its inherent limitations for the specific case of RMA-2 application to long reach floodplain environments.

5.2.3 Establishment of a finite element mesh

On the basis of previous applications of RMA-2 to large-scale floodplain and other environments (King and Roig, 1988; Gee *et al.*, 1990), broad criteria for the establishment of the mesh can be set out. These criteria, along with the improvements to the physical representation by finite elements for floodplain systems suggested in Section 3.3.2, minimize mesh configurations that produce sharp changes of slope or situations where large volumes of water flow into small elements. Three such properties of mesh design were seen as desirable for inclusion in any finite element discretization of the River Culm. Firstly, strips of elements running on either side of the channel were added to model the effects of bankside roughness and the presence of bankside levees. Secondly, the mesh was configured so that its longitudinal lines tended to run parallel to the channel, thus ensuring floodwater advance and recession across the floodplain was modelled in a smooth fashion. Lastly, lateral lines were set up to cross the reach perpendicular to the mesh long axis in order to minimize the front width used in the numerical solution. This has the effect of making the solution more efficient. In addition, structures to control the flow of water into and out of the finite element mesh were implemented at the up- and downstream extremities of the grid. These control structures ensured inflow and outflow occurred parallel with the mesh sides, thereby improving solution stability.

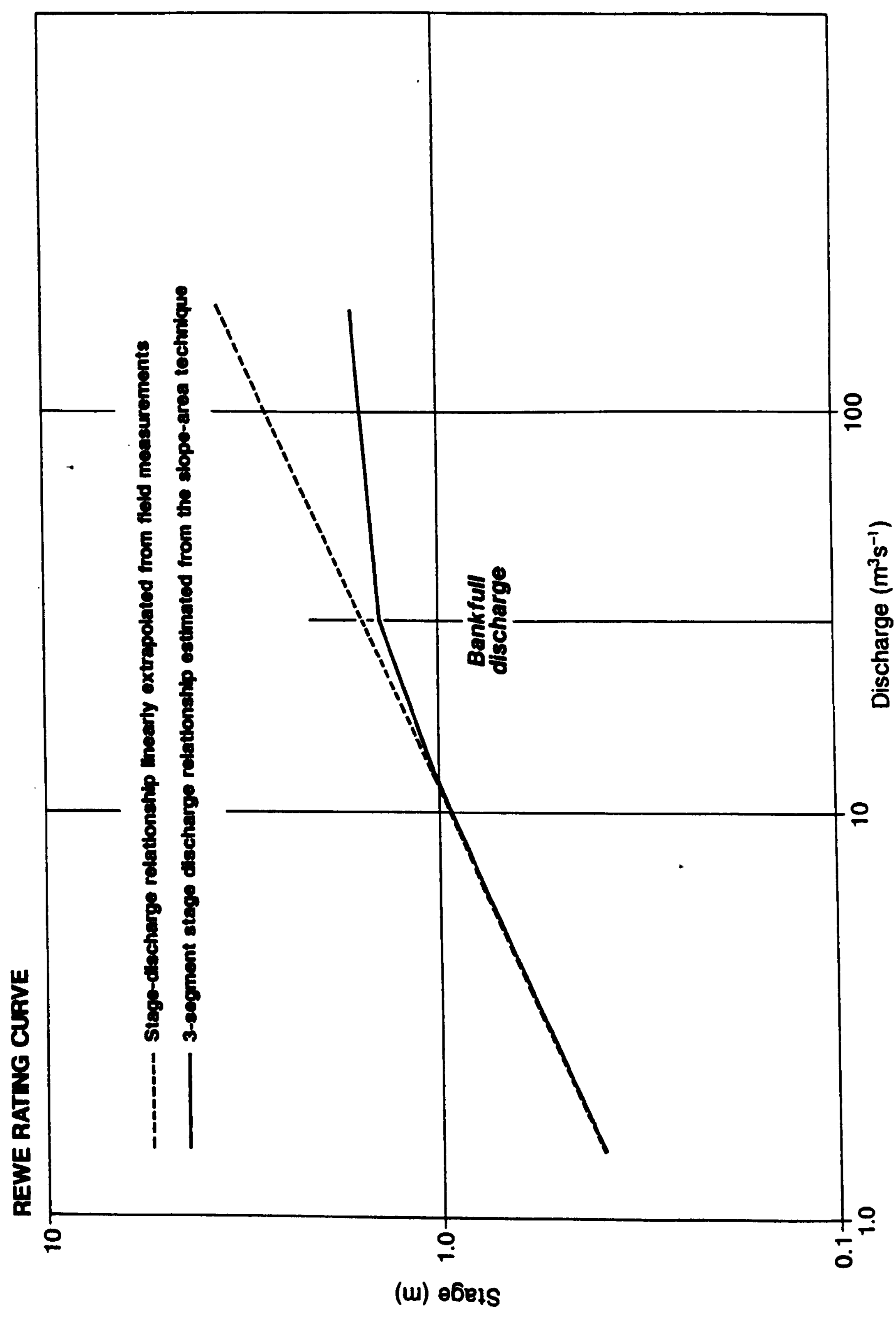


Figure 5.3: Rating curves established for the River Culm at Rewe.

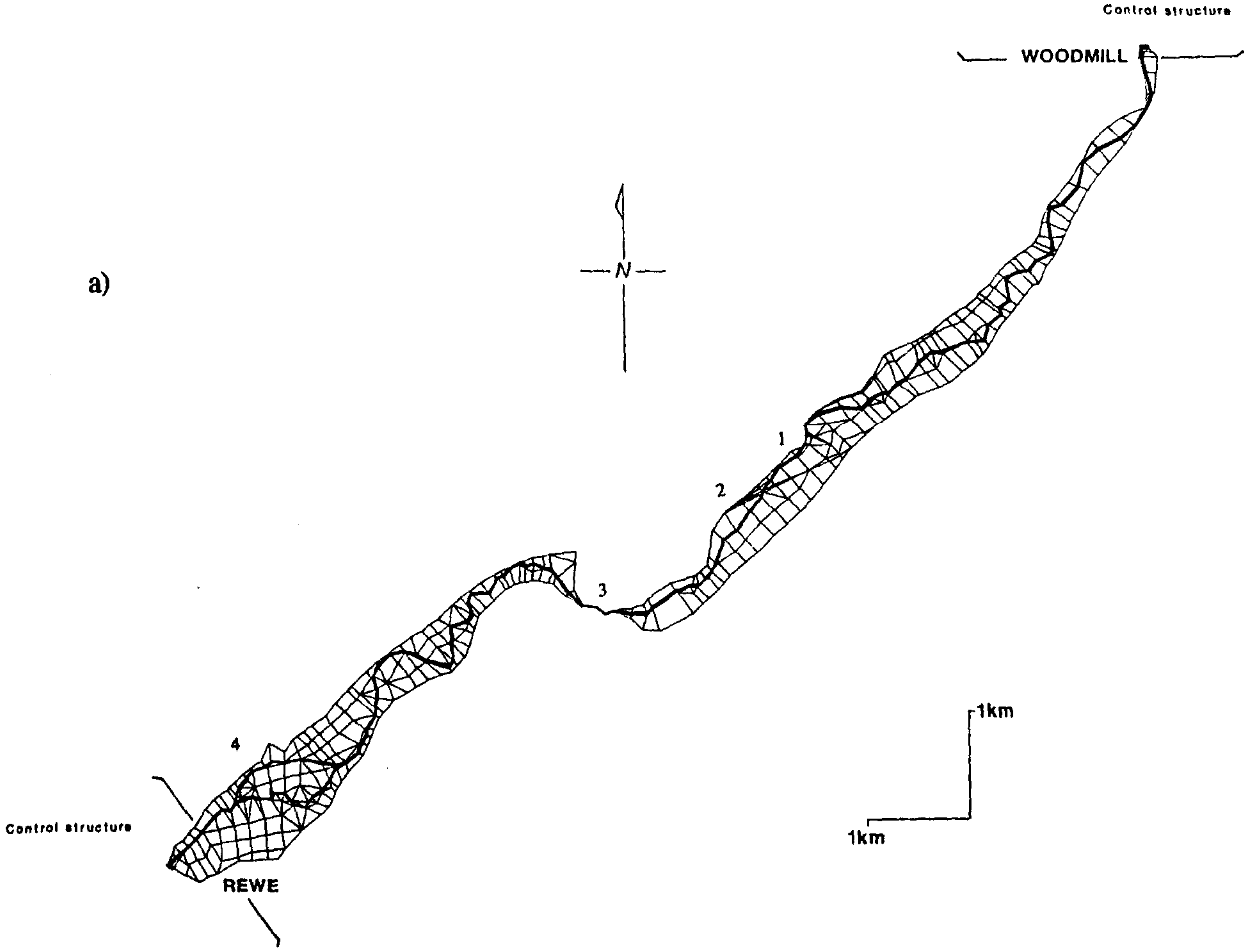
The finite element mesh resulting from these criteria consisted of 1090 elements and 3655 nodes. Mesh configuration also required that certain specific physical features of the floodplain had to be included (features 1 - 4 on Figure 5.4). Mill races (features 1 and 3) are represented as channelized reaches, with the flow at Silverton contained entirely within the channel and no floodplain elements required. The railway embankment (feature 2) is modelled as a barrier across the floodplain, breachable by high flows. This creates a pond of low velocity water and the funnelling of flow through a bridge near the right hand bank. The 1 km long bifurcation (feature 4) at the downstream end of the reach creates a large and topographically complex floodplain island where flows are assumed to be equally divided between the two parts of the channel.

In order to establish the degree of topographic information that could be specified in the finite element discretization for long reach floodplain applications, the above construction rules were implemented using all available topographic data. This was designed to achieve an overspecified, and therefore unstable, topographic discretization. This excess representation could then be relaxed until stability was achieved, thus generating the 'best-fit' finite element discretization for the problem in hand. Although the operational rules developed from this procedure will be site specific, the topographic complexity of this reach is exceptional. We can therefore be reasonably confident that operational rules developed for this reach will be more generally applicable.

5.2.4 Model development: mesh refinement and topographic resolution

Having established an initial mesh on the above basis, test runs were undertaken with a steady flow simulation in order to develop initial conditions for dynamic simulations. As anticipated in Section 5.2.3 these proved to be unstable, with poor continuity and convergence statistics. Such instabilities have been discussed in terms of the sensitivity analysis outlined in Chapter 4, with similar criteria being applied here to simulations involving the entire study reach. For stable solutions, the continuity, as measured by flow crossing a line that transects the entire region of fluid movement, was defined as being within ±2% of that entering the reach. Similarly the convergence, or relative change in system state variables between iterations, had to be within the range ±0.0001-0.2% to be classified as stable. In the case of unstable solutions the iterative numerical procedure failed to converge on a single value,

a)



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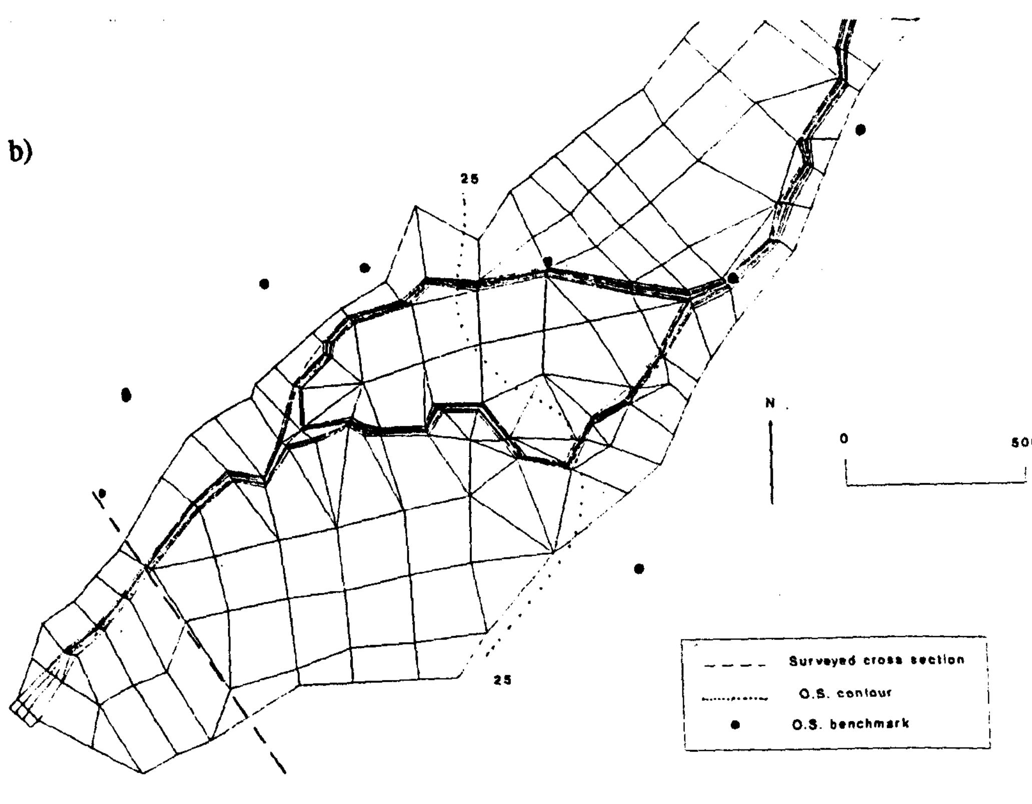


Figure 5.4: a) Finite element mesh configured for an 11 km reach of the River Culm, Devon between Woodmill and Rewe showing inflow and outflow control structures. b) The relationship between topographic data sources and mesh construction for the lower portion of the finite element mesh shown in Figure 5.4a.

giving instead greater estimates of system state variables as iterations progressed. Thus the convergence statistic tended away from zero exponentially. It was quickly established that this phenomenon only occurred for specific portions of the mesh. In this event the set of results calculated by the model for each node could be interrogated and sections causing these problems identified by the unrealistically high velocity vectors and depths predicted. The mesh in these areas was found to be characterised by one or more of the following features:

i) areas where large volumes of water flow into small elements,
ii) areas of steep lateral or longitudinal slope,
iii) sharp changes in the direction of flow,
iv) the four specific features identified on Figure 5.4.

These therefore represent regions of excess inclusion of topographic detail into the finite element discretization. To overcome this the representation of channel planform, channel cross-section, floodplain topographic resolution and longitudinal slope was refined. This was achieved by systematically identifying the unstable areas in the above fashion, isolating the particular set of problem features present and then gradually relaxing the mesh representation until a stable solution was found. The effects of these procedures are shown in Figure 5.5. This area represents the mesh section most radically altered by the smoothing process. As a result of these developments a number of guide-lines have been developed for use in future applications. These are:

i) Channel planform - an upper limit of approximately 110° has been found to apply to the representation of meanders. This criteria then sets constraints on the minimum length of channel element that it is possible to apply.

ii) Channel long profile - the upper limit for slopes is a 2 m drop per kilometre of channel section.

iii) Lateral slopes - the upper limit of representation here is a 3 m change in elevation between adjacent nodes.

iv) Floodplain topographic representation - this should represent overall form (topographic structures in the size range 10-100 m) rather than micro-scale variations (topographic structures in the size range 1-10 m) and is in

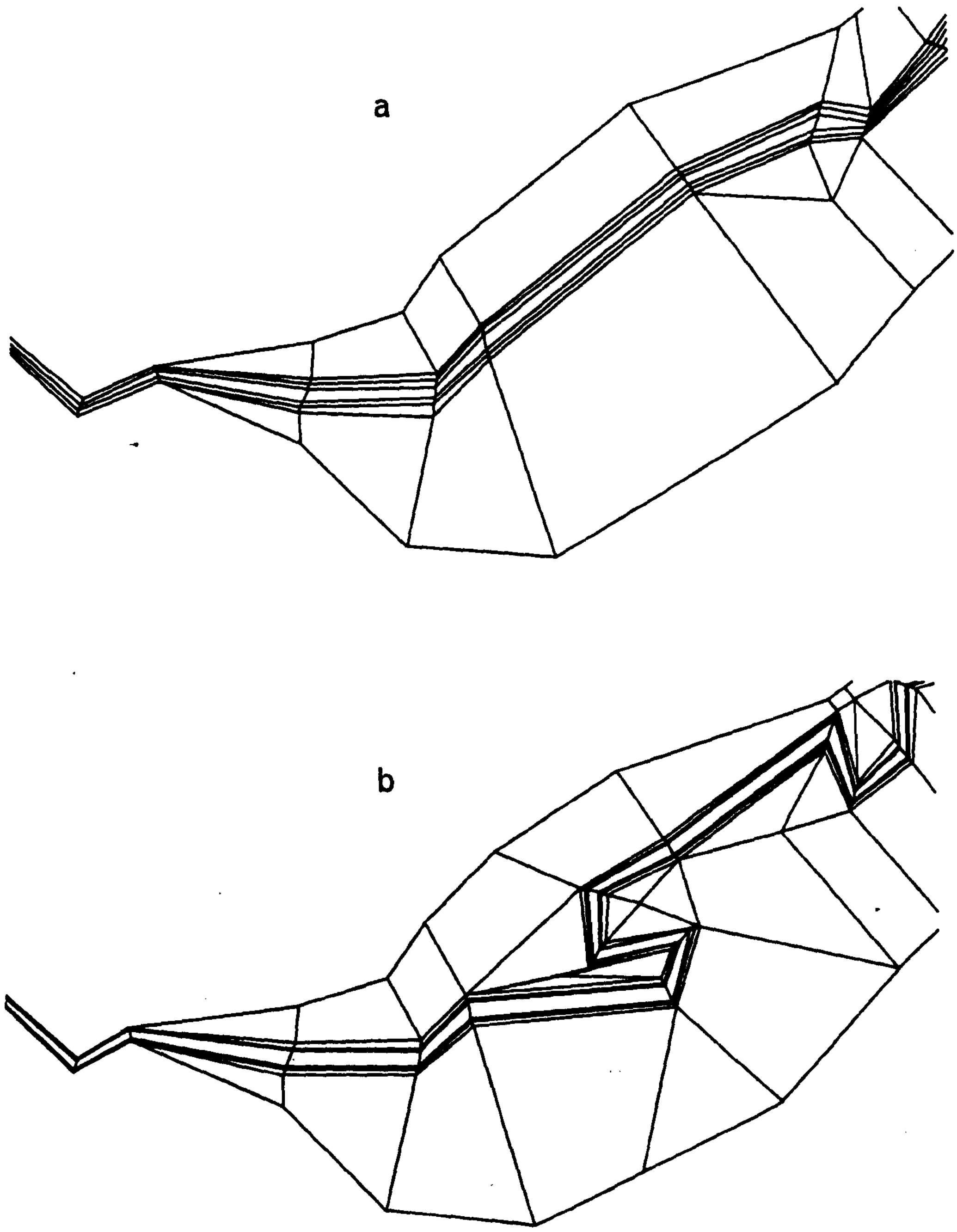


Figure 5.5: Development of mesh configurations for RMA-2 floodplain applications; a) final operational mesh, b) initial configuration using maximum available topographic data. This represents the section most radically altered by the mesh development process.

accordance with the stated objective (see Chapter 1) of simulating flow features at this scale.

Independent testing of these operational rules will be undertaken in Chapter 8 for a further application of the enhanced RMA-2 scheme.

5.3 Initial validation of simulated process behaviour

With high spatial and temporal resolution environmental models complete validation of simulated process behaviour may not be possible. Data may be lacking to fully perform the task and additionally, complex simulations may produce data sets that are too large to be fully examined. Furthermore, most distributed modelling studies only attempt prediction validation at a small number of specific nodes, usually at the downstream outlet of the modelled system. A positive validation of the model for this point is used to infer an accurate simulation of distributed process behaviour for the entire system. Despite obvious inadequacies, however, it is common for modelling studies to only consider evaluation methodologies of this type (Howes and Anderson, 1988). In this study, evaluation of simulated process behaviour at the downstream system outlet forms one part of a larger evaluation procedure including rigourous mathematical model validation, computer model verification and validation of simulated process behaviour for the entire reach. Reliance on the validation of model predictions at the downstream system outlet as the sole means of assessment has thus been circumvented, although this still remains an essential test for any model and will therefore be considered in detail here.

This Section will discuss three separate aspects of the validation of simulated process behaviour at the downstream system outlet for the refined RMA-2 model. Firstly, model behaviour during dynamic simulations of real flood events is compared to observed process data for discharge, stage and velocity for the River Culm. Secondly, on the basis of conclusions drawn in the course of this validation exercise, a series of refinements to model structure, with the potential to improve model predictions, will be implemented and assessed. Finally, the particular case of model response to low flow inputs will be considered to confirm the conclusions regarding this type of simulation drawn in Chapter 4.

The boundary conditions and test data used to develop these model simulations are summarised in Table a.

Run (by Figure number)	Upstream boundary condition	Downstream boundary condition	Predicted variable	Test data
5.7	Discharge + control structure	HYMO3 rating curve + control structure	Stage at Rewe	Stage at Rewe
5.9	Discharge + control structure	HYMO3 rating curve + control structure	Discharge at Rewe (area x velocity)	Rewe discharge assessed using the linearly extrapolated Rewe rating curve
5.10	Discharge + control structure	HYMO3 rating curve + control structure	Channel discharge at Rewe (area x velocity)	Rewe discharge assessed using the channel-only Rewe rating curve
5.11	Discharge + control structure	HYMO3 rating curve + control structure	Discharge at Rewe (area x velocity)	Rewe discharge assessed using a rating curve estimated by the slope-area technique
5.12a	Discharge + control structure	HYMO3 rating curve	Stage at Rewe	Stage at Rewe
5.12b	Discharge + control structure	Stage	Stage at Rewe	Stage at Rewe
5.13	Discharge + control structure	Stage	Discharge at Rewe (area x velocity)	Rewe discharge assessed using a rating curve estimated by the slope-area technique
5.14 (extended mesh)	Discharge + control structure	HYMO3 rating curve + control structure applied at Stoke Cannon	Stage at Rewe	Stage at Rewe
5.15	Discharge + control structure	HYMO3 rating curve + control structure	Discharge at Rewe (area x velocity)	Rewe discharge assessed using a rating curve estimated by the slope-area technique

Table a: Boundary conditions, predicted variables and test data used to generate the model simulations detailed in Chapter 5.

Hydrograph simulations were carried out with two flood events (Figure 5.6) using a 0.5 hour time step. These simulations were undertaken on a MIPS M/2000 computer and typically required between 15 and 50 minutes of c.p.u. time. The chosen floods were a simple, single peaked hydrograph (event **a** on Figure 5.6) and a double peaked event (event **b** on Figure 5.6), representing the most complex flow conditions that the model would be required to deal with. In terms of event magnitude, the latter consists of 1 in 1 and 1 in 5 year recurrence interval events as measured at Cullompton, while the single peaked flood represents an event frequency of 1 in 1 year.

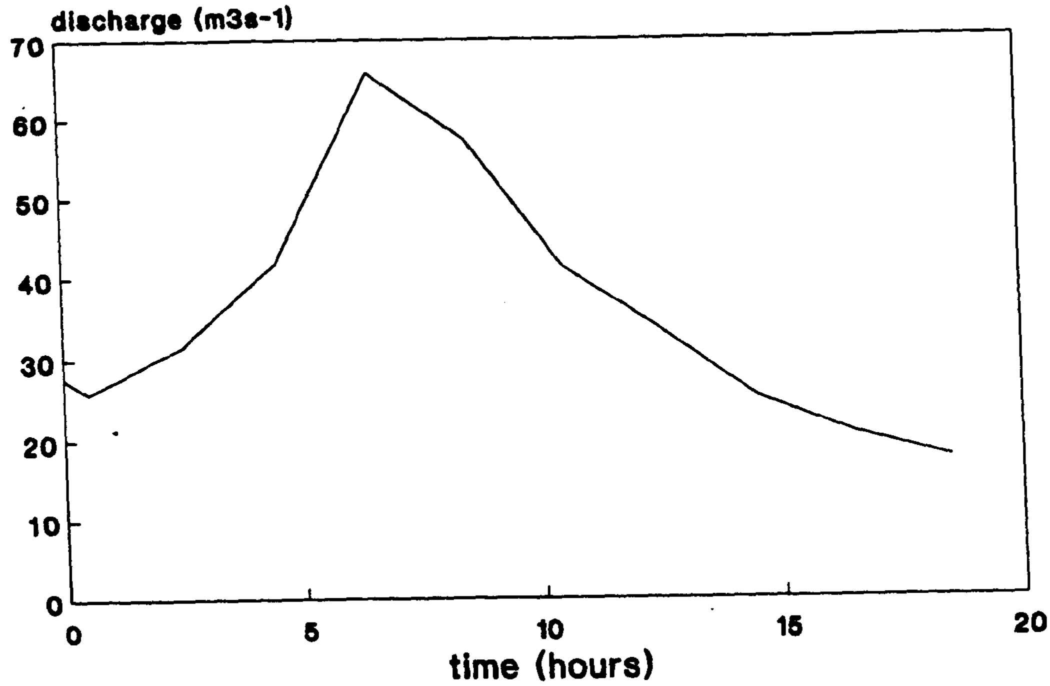
In order to initially test simple calibration schemes for RMA-2, floodplain boundary friction values of $n = 0.045$ and $n = 0.1$ were used in these simulations. This is broadly in line with the range of n values suggested in a study of floodplain friction factors by Acrement and Schneider (1984). Floodplain boundary friction has been highlighted as a potential calibration parameter by previous studies of model sensitivity (Baird and Anderson, 1990), but was also chosen due to the relatively good documentary record (see for example Chow, 1959; Acrement and Schneider, 1984) of typical values which includes details of its spatial and temporal variation. This was used to ensure that the selected calibration schemes remained physically realistic.

Two events only were simulated, as for complex modelling schemes, such as RMA-2, it is more scientifically valid to rigourously test a small number of flood events, than attempt a larger number of simulations that can only be superficially analysed. Predictions from these simulations were compared to observed process data. Of these process observations, the stage time series for the Rewe gauging site is subject to the least degree of measurement error. Applying a HYMO3 derived stage discharge rating curve to these figures to obtain the discharge compounds this, introducing errors of ±15% (see Chapter 3). The discharge record for this application therefore provides a less clear validation of the model performance. However, as previously suggested, such testing is necessary if the model's total performance is to be analysed.

5.3.1 Validation of stage predictions

Predicted stage data for the Rewe gauging site were compared to observed values for both events (Figure 5.7). Stage predictions show good correspondence to observations of the physical system, although the degree of flow attenuation predicted by the model over the reach does not fully match observations, with model results

a)



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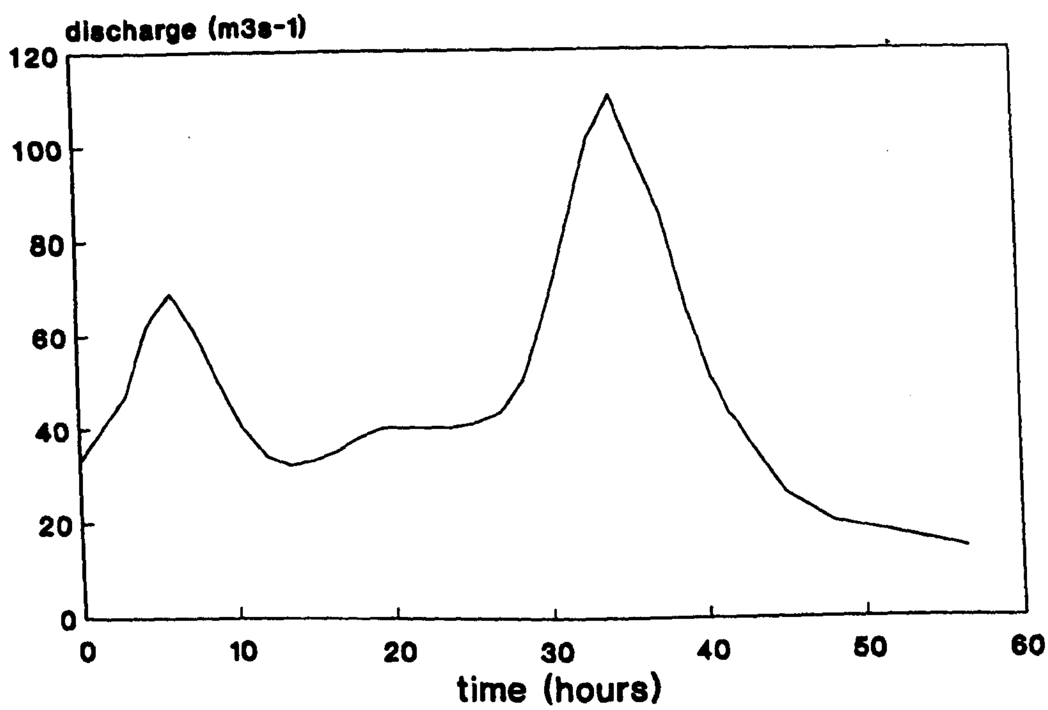


Figure 5.6: Flood events used in RMA-2 validation; a) single peaked 1 in 1 year recurrence interval event, b) double peaked event consisting of 1 in 1 and 1 in 5 year events.

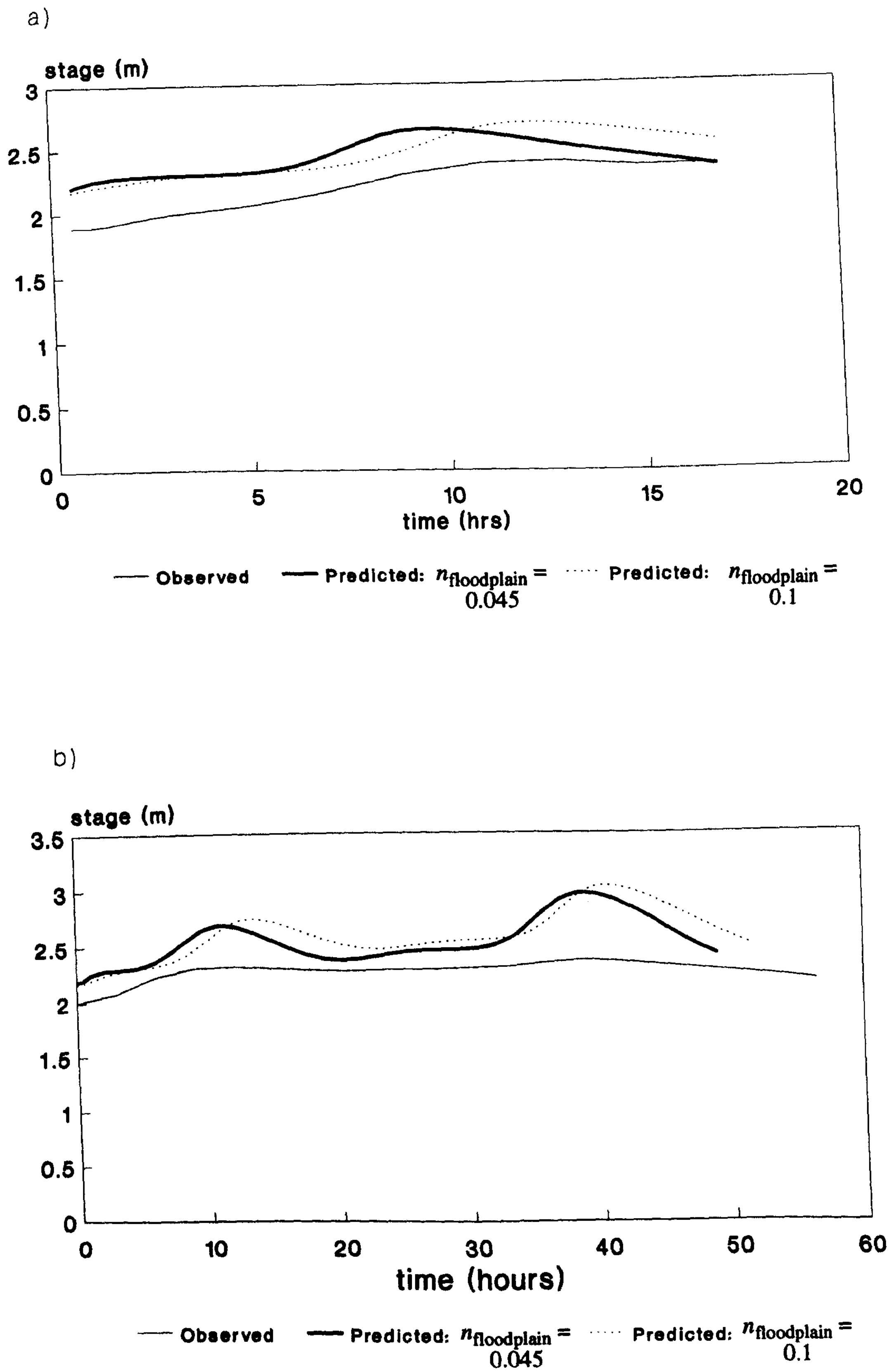


Figure 5.7: A comparison of observed and predicted stage data for the Rewe gauging station, for events **a** and **b** in Figure 5.6, using $n_{floodplain}$ = 0.045 and 0.1

overestimating observed stage for flood peaks. The timing of predicted peak stage is still, however, reasonable. Interrogation of the results data set showed that overprediction of peak stage was locally confined to an area immediately adjacent to the downstream control structure, a fact clearly evident from plots of predicted water depths in this region (Figure 5.8). This is believed to be induced by the impact of the downstream control structure on the flow field. By creating an effective constriction, outflow from the finite element mesh is impeded and flow locally backs up onto the floodplain. This appears to be the only mechanism capable of explaining these model results.

It must be stressed that this local error in predicted stage will also apply to discharge, although the effect here may be less obvious due to the greater errors inherent in discharge estimation. Despite this structurally induced error, **the model's initial ability to reproduce aspects of the observed stage record is good.**

5.3.2 Validation of discharge predictions

Section 5.2.2 suggests three methods of developing the available stage data for the Rewe gauging station in order to interrogate the discharge record. Firstly, model predictions were compared to discharge data obtained by applying the original, linearly extrapolated rating curve (Figure 5.9). Although a large discrepancy occurs between observed and predicted outflow, two points are evident. Firstly, there appears to be a volumetric error in the observed discharge record. For example, observed outflow for the double peaked event shown in Figure 5.9a, is only 52% of observed inflow, compared to a more realistic 94% for predicted outflow. Secondly, for both events the model provides a good fit to the data until a discharge of 26 m^3s^{-1} is reached; this coincides approximately with bankfull flow for the Rewe gauging station. These results partially validated the model's operation and confirmed that a linear extrapolation of the in-bank rating curve was inadequate for estimating out-of-bank flow.

Isolating the channel flow component and comparing this to the predicted value proved to be a more satisfactory method of assessing the model's performance (Figure 5.10). With a floodplain Manning's *n* coefficient of 0.045, RMA-2 predictions underestimated the channel flow component. However, increasing the value of this parameter to 0.1 for the double peaked flood event led to a better fit between

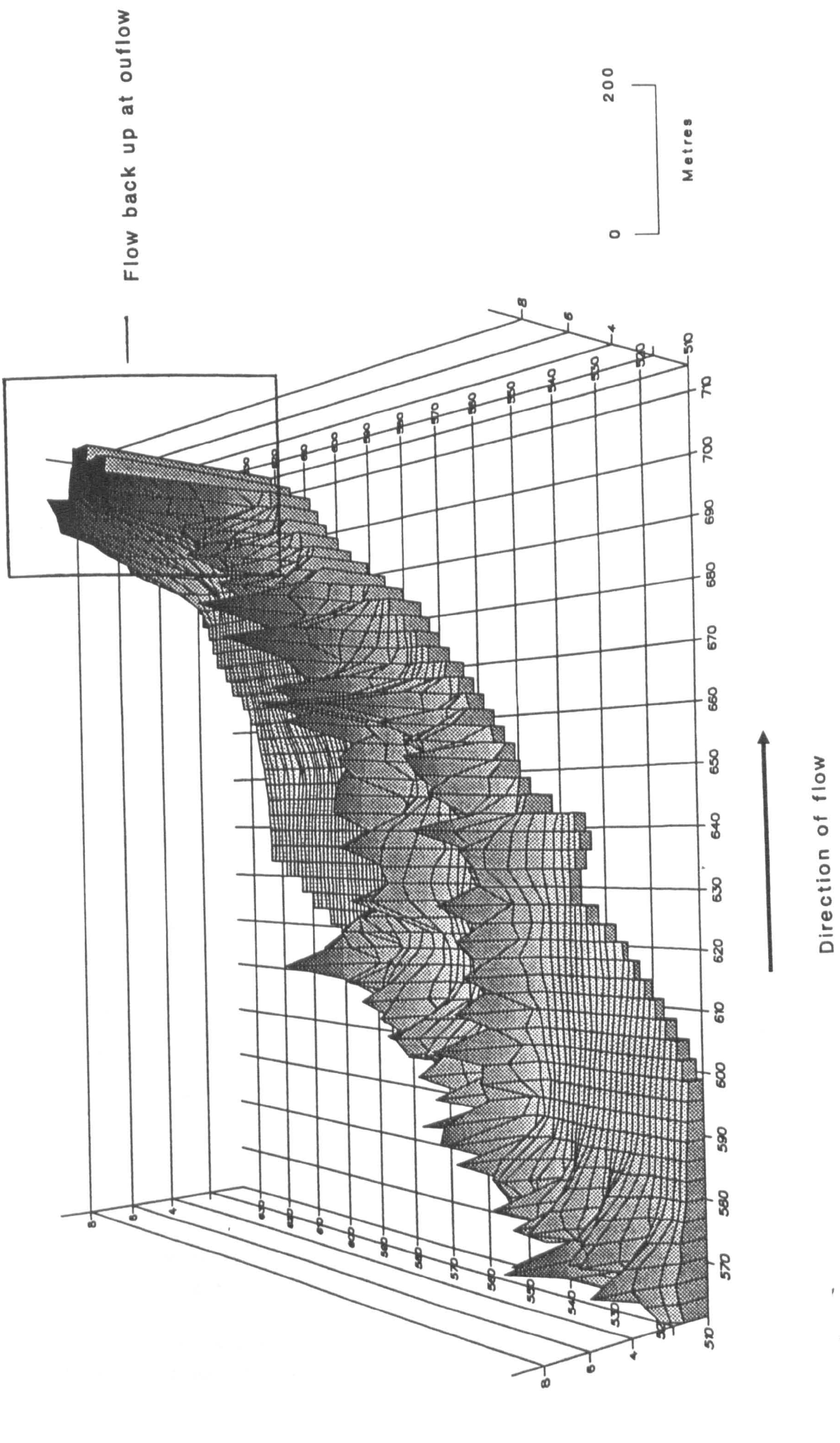


Figure 5.8: Three dimensional plot of inverted water depth for the area of mesh including the channel bifurcation at Rewe. This diagram shows localized backing up of flow onto the floodplain caused by the downstream control structure.

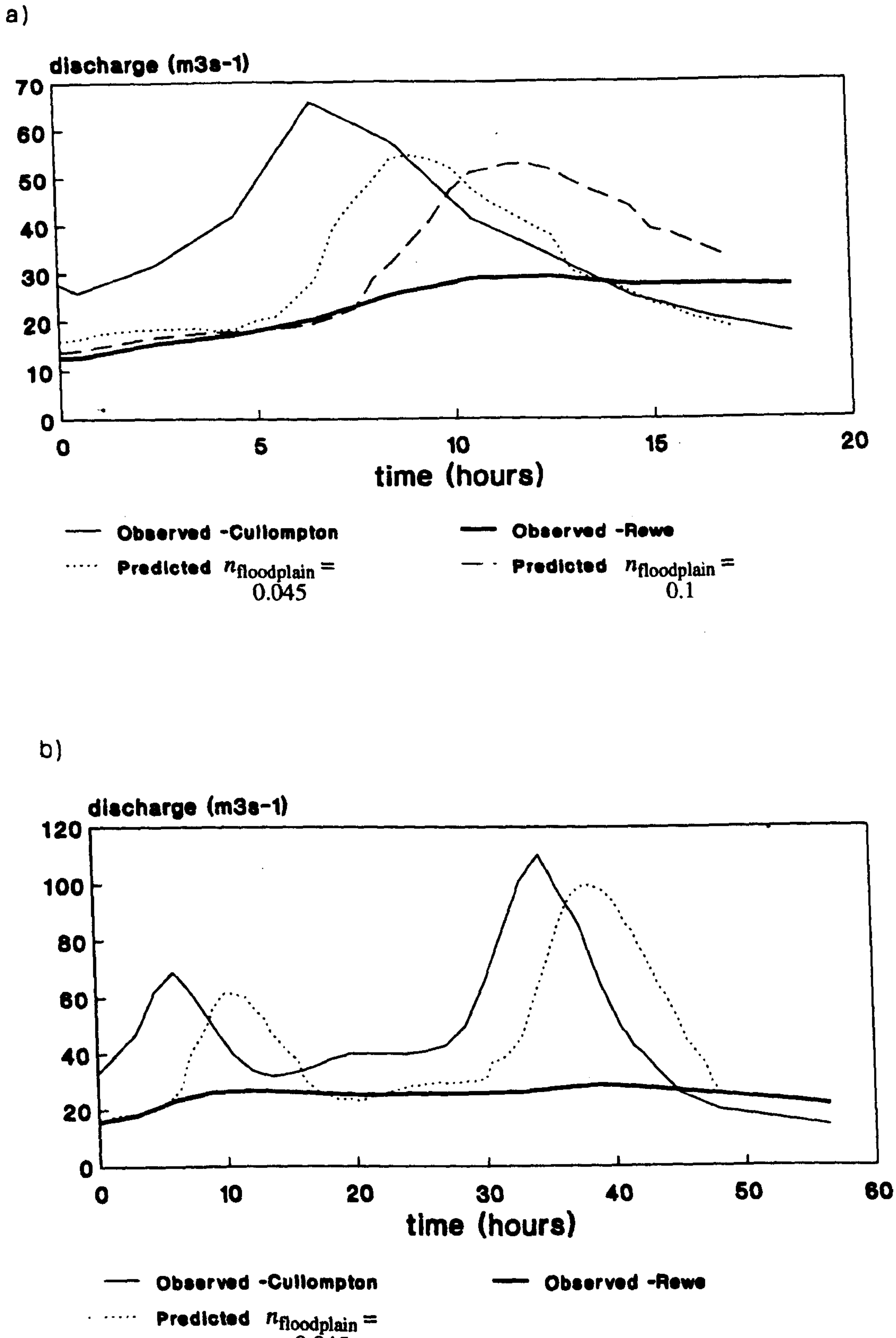
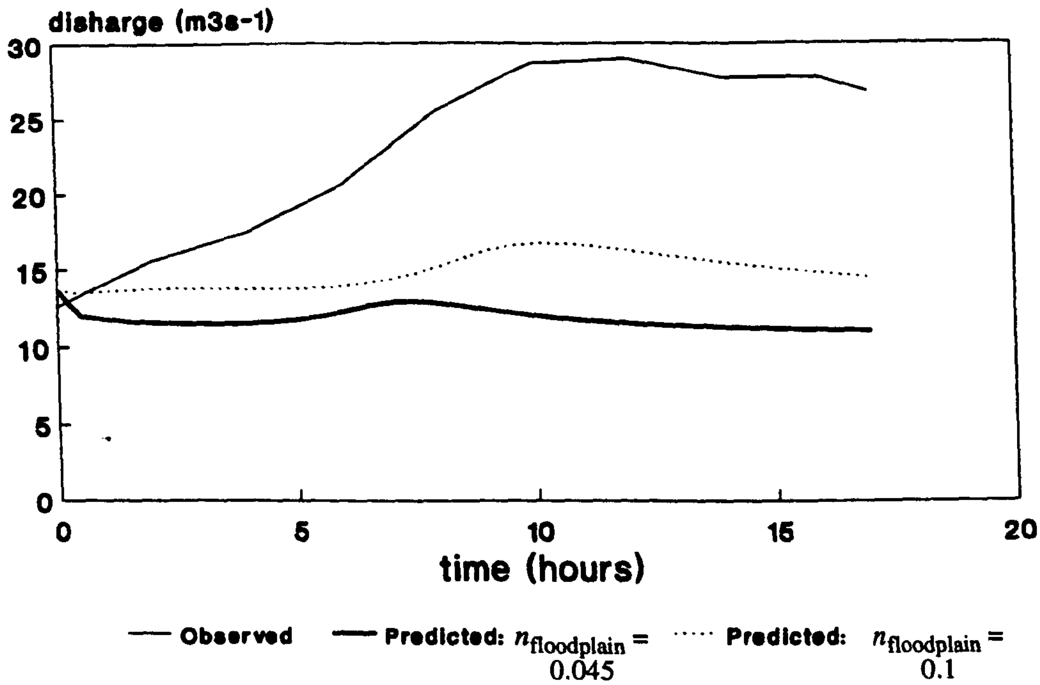


Figure 5.9: A comparison of observed and predicted discharge data for the Rewe gauging station, assessed using a linearly extrapolated rating curve, for events a and b in Figure 5.6, using $n_{floodplain}$ = 0.045 and 0.1.

a)



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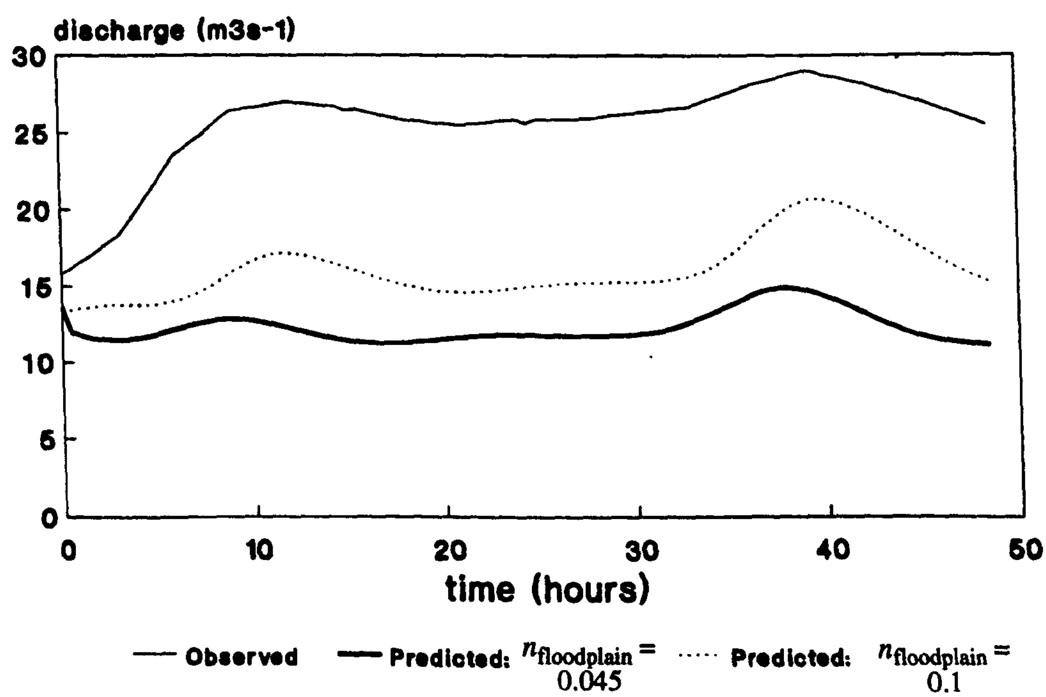


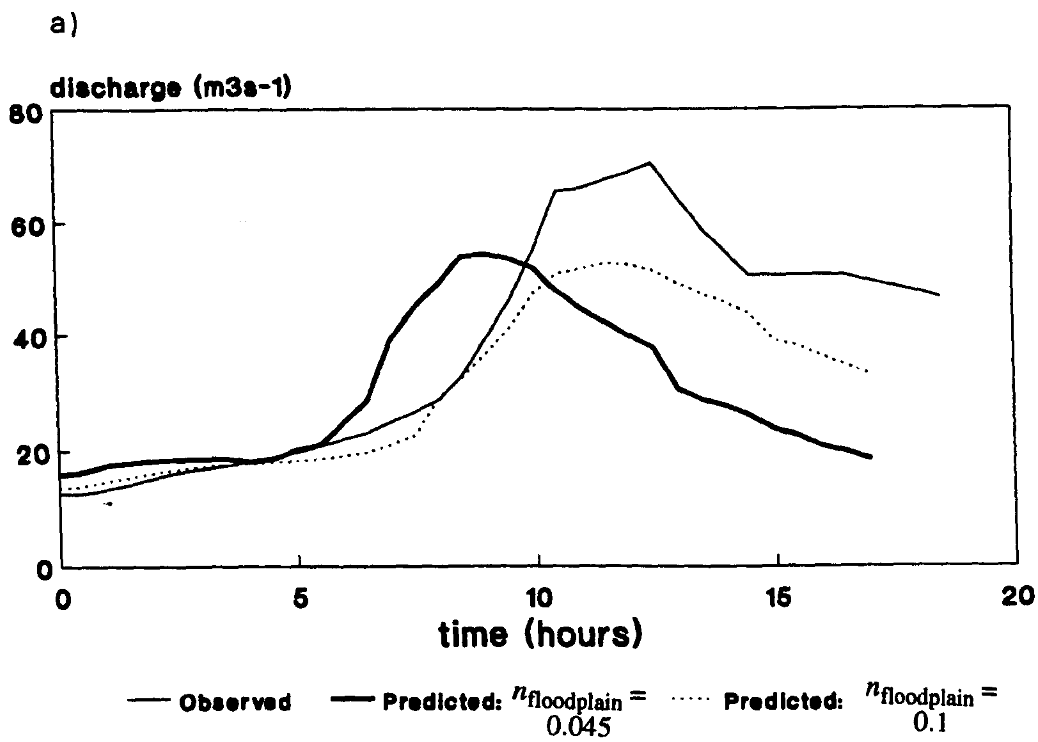
Figure 5.10: A comparison of observed and predicted channel discharge data for the Rewe gauging station, assessed using a linearly extrapolated rating curve, for events a and b in Figure 5.6, using $n_{floodplain}$ = 0.045 and 0.1.

predicted and observed data, although underprediction is still apparent. This is potentially explained by the observation that errors in this rating curve could be of the order of ±25% (Martin and Myers, 1991). We cannot therefore be certain whether the discrepancy between observed and predicted hydrographs is due to the modelling scheme or the assumptions of the rating curve estimation method. In particular, a natural channel could never be a perfectly uniform rectangle and also the estimate of observed channel discharge fails to take into account any possible interaction between main channel and floodplain flows. Despite these factors, the simulations provide an indication that the channel/floodplain flow ratio simulated by the model is approximately correct.

Re-evaluating the Rewe stage data with the three segment rating curve established by the slope-area technique (Figure 5.11) led to a much better correspondence between observed and predicted discharge. The timing of predicted peak discharge is improved in both cases by increasing the floodplain roughness coefficient to 0.1. In addition, a much closer fit to the pre- and inter-peak discharge is evident in these simulations. It is noticeable that the model underpredicts peak discharge for the single peaked flood (event **a**), while the opposite pattern is true for the second flood simulated (event **b**). This possibly reflects a variation in rating curve error with stage and indicates the potential discrepancies due to the rating curve estimation method. Further supporting evidence for this view is provided by continued volumetric errors in the observed record, which mirror the prediction uncertainties identified above. Despite this, we can state that model predictions lie within the error band of the rating curve estimation method. We are thus able to validate the model's discharge predictions at its downstream **external** boundary.

5.3.3 Validation of velocity predictions

No adequate data exists with which to validate RMA-2 velocity vector predictions. This is due to the difficulty of collecting synoptic, depth averaged velocity data for a sufficiently large proportion of the mesh to allow meaningful comparisons to be made. Despite this it is possible to state that as we are able to reasonably predict both stage and discharge, it follows that overall, the velocity vector predictions are approximately correct. However, it is impossible to make any statements concerning detailed velocity distributions within the flow field, although it is possible to analyse



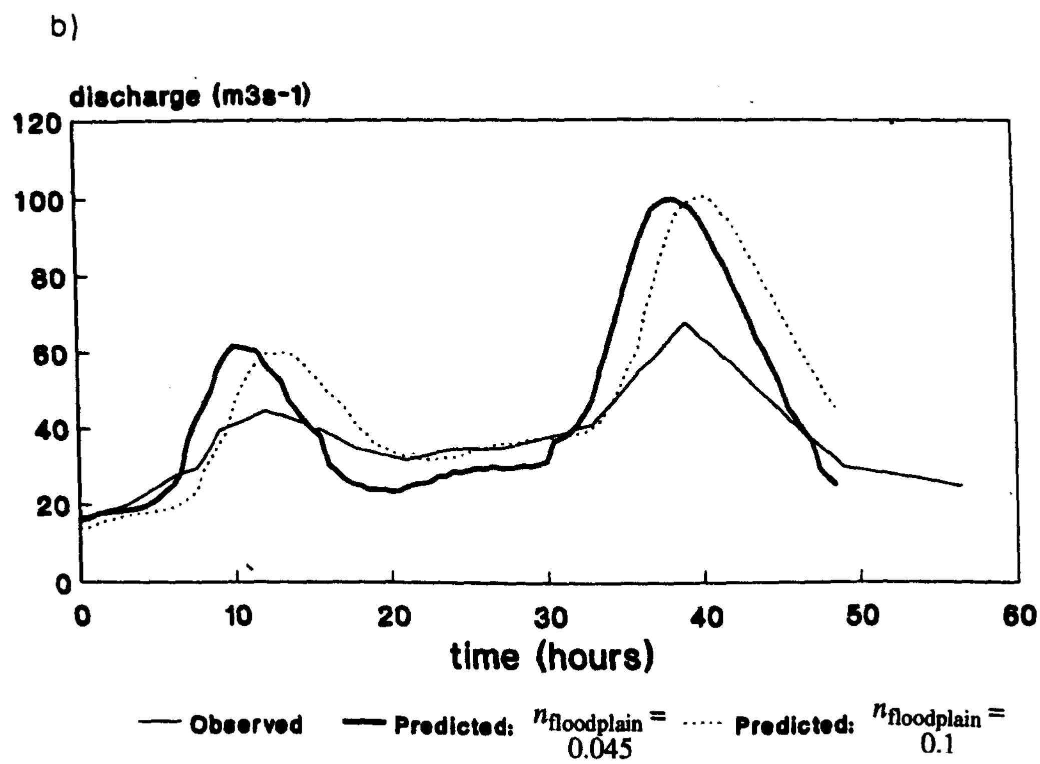


Figure 5.11: A comparison of observed and predicted discharge data for the Rewe gauging station, assessed using a rating curve estimated from the slope-area technique, for events a and b in Figure 5.6, using $n_{floodplain}$ = 0.045 and 0.1.

simulated velocity vector fields theoretically in terms of the known flow features of the physical system (see Section 8.2).

5.3.4 Simulation refinement

Three refinements to the basic model formulation assessed above were tested in an attempt to improve model predictions. These were:

i) removal of the downstream control structure,

ii) implementation of a stage hydrograph boundary condition at the downstream mesh boundary on this refined mesh,

iii) extension of the mesh.

The results of implementing changes (i) and (ii) are given in Figure 5.12, simulated with $n = 0.045$. Removal of the downstream control structure slightly improves both the magnitude and timing of the stage predictions. However, overprediction of stage still occurs. Greater improvement in predictive ability may have been expected from this refinement, yet the control structure has not been replaced with any better solution. Thus little improvement is shown. This therefore demonstrates a need to develop operational rules for control structure construction that are appropriate to long reach scale applications. When the mesh formulation with the control structure removed is combined with a stage boundary condition, predictions are much improved and the degree of attenuation in the physical system is reproduced to a high degree of accuracy. Discharge predictions with such a boundary condition are however, significantly above observed values (see Figure 5.13). This implies that although the velocity vector field is not accurately reproduced, such simulations may be appropriate for certain applications where it is only necessary to predict water surface elevations.

The finite element mesh detailed in this Chapter has been extended in order to examine the model's application potential for predicting flood risk in the vicinity of a small village on the floodplain 3 km downstream of Rewe, for which a significant flood hazard exists. This model application is discussed in detail in Chapter 8, however it is appropriate at this point to consider the improvement in Rewe stage

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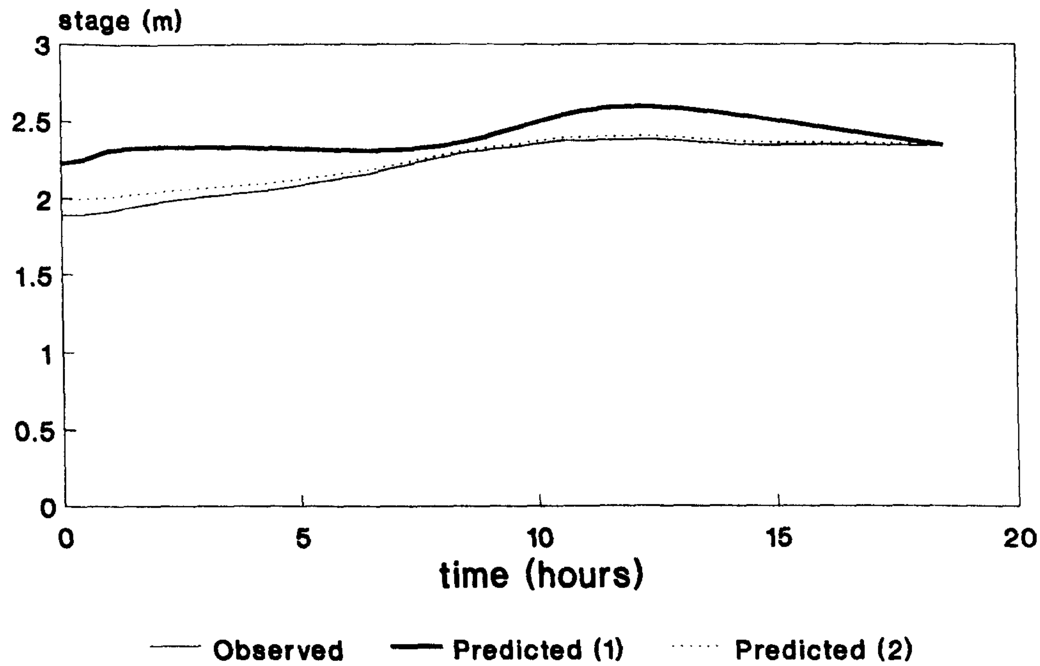

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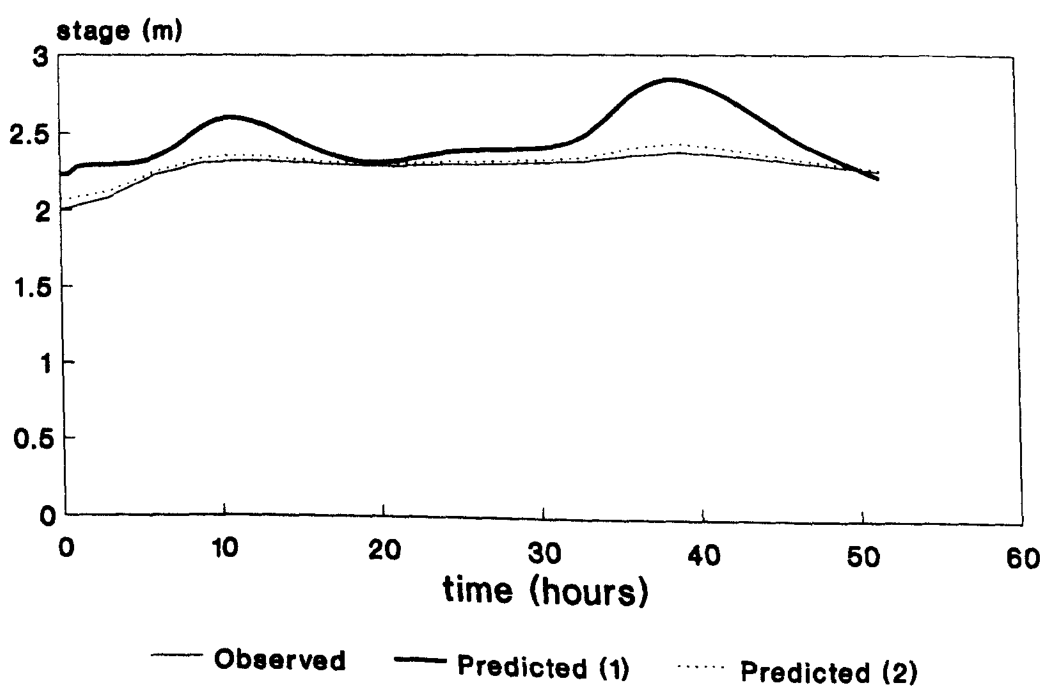
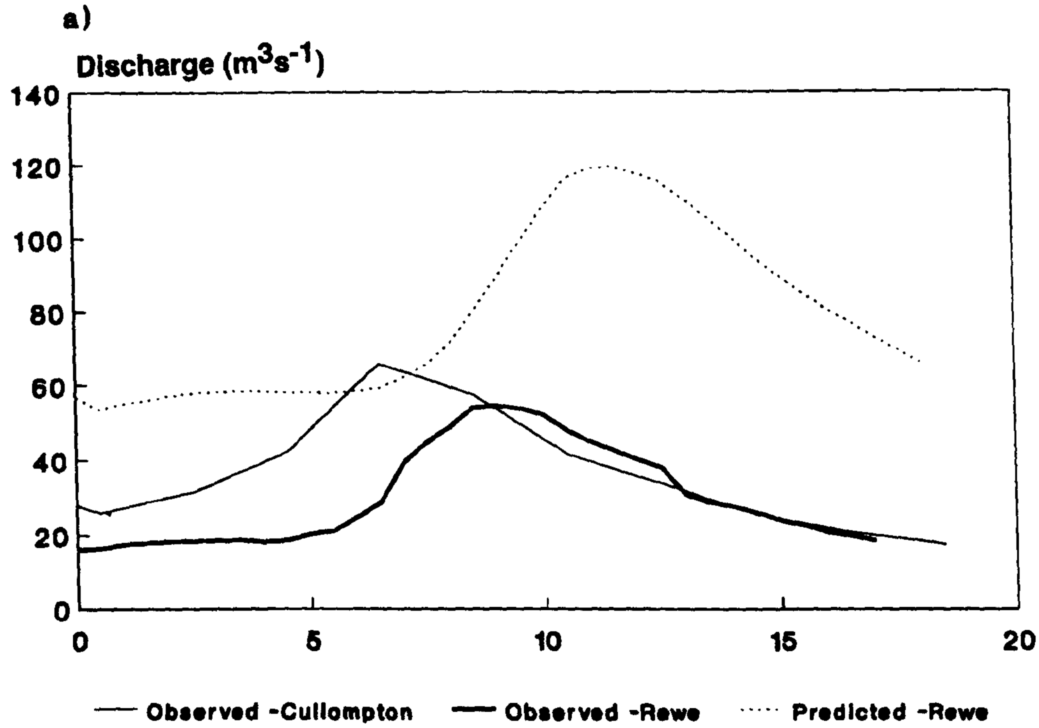


Figure 5.12: A comparison of observed and predicted stage data for the Rewe gauging station for events **a** and **b** in Figure 5.6, developed on the basis of two mesh refinements. 1) No control structure, 2) no control structure and a stage hydrograph boundary condition.



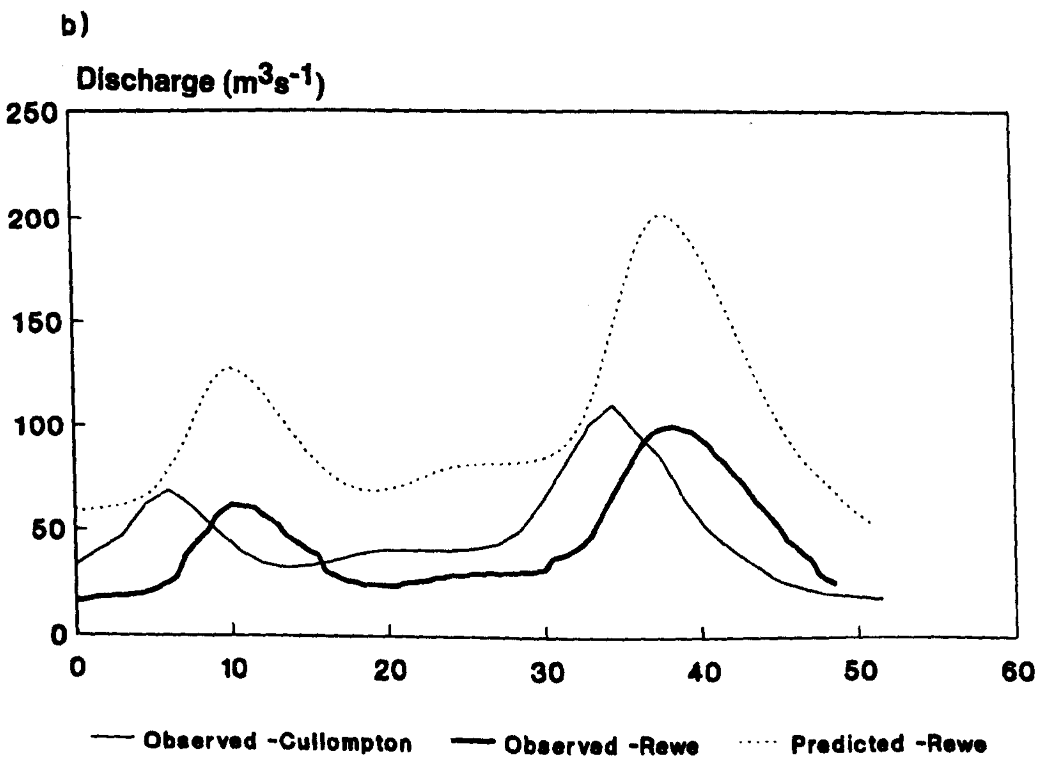


Figure 5.13: A comparison of observed and predicted discharge data for the Rewe gauging station for events a and b in Figure 5.6, using a stage hydrograph boundary condition and no control structure.

predictions resulting from this extension (Figure 5.14), as this effectively displaces the constriction problem downstream. The extended mesh simulation shown in Figure 5.14 demonstrates the model's ability to accurately reproduce the level of flow attenuation displayed by this particular system, marginally underestimating the observed stage. These predictions were improved with floodplain roughness of n = 0.1 and it is likely that further improvements to this calibration could be made if necessary. This provides confirmation that overestimation of stage and discharge is locally confined to the region around the downstream control structure and that predictions upstream from such localized effects will closely fit observed data.

This has a number of implications for modelling strategy. Just as application specific calibration schemes can be devised, physical representation by the finite element scheme can be altered to achieve particular objectives. This confirms the view taken in Chapter 3 that at the current stage of model development it is more fundamental to undertake refinement of the physical representation, than implement alterations to the physical model that are potentially less significant. Two modelling strategies, unique to floodplain environments, have consequently been suggested on the basis of varying data availability that can be used to meet application-specific objectives. Firstly, using a stage time series as the downstream boundary condition, stage predictions can be produced to a high degree of accuracy, if it is unimportant to predict other aspects of the flow field. Secondly, for applications where the data availability is poor, or where the total flow field is of concern, a rating curve boundary condition can be generated and implemented. In this case the mesh should be extended a short distance (0.5 km) below the normal downstream boundary.

Validation of simulated process behaviour has also demonstrated that refinements to model predictions are achieved with a floodplain boundary friction coefficient of n = 0.1. This is consistent with conclusions drawn in previous modelling studies (Gee *et al.*, 1990) and with field observations of Manning's n values (Acrement and Schneider, 1984). It therefore appears possible to operate the enhanced RMA-2 scheme for long reach floodplain applications at an extremely low level of calibration. This reduces the potential that physically unrealistic calibration schemes can be constructed and, in addition, will reduce the time and resources required to satisfactorily complete any modelling study.

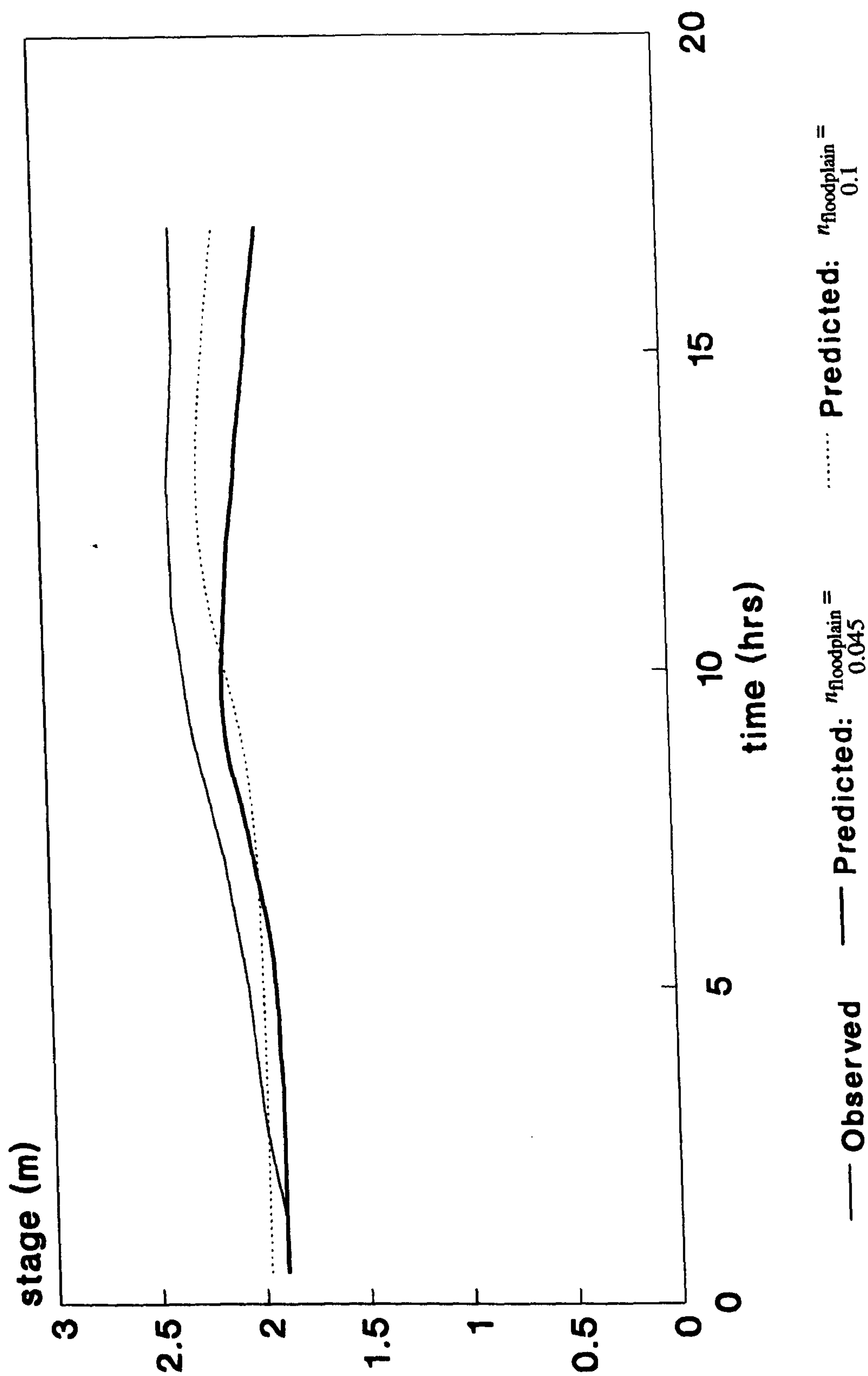


Figure 5.14: Improved prediction of stage hydrographs for the Rewe gauging station with an extended finite element mesh for flood event a in Figure 5.6 and $n_{floodplain}$ = 0.045 and 0.1

5.3.5 Model response to low flow inputs

An attempt was also made to validate the refined RMA-2 model against very small flood events, with frequencies << 1 in 1 year. The results of one such simulation are presented in Figure 5.15. This shows that for flows at, or below, bankfull discharge the model provides a poor fit to observed data and eventually becomes unstable. This was felt to be due to the large numbers of elements entering and leaving the solution during such simulations, under control of the wetting/drying algorithm parameters. It is suggested that a solution to this problem could be found if the resolution of elements in floodplain areas was dramatically increased. However, this would lead to a computationally inefficient model, with solution resolution concentrated in regions of low state variable gradients. We may therefore confirm the conclusion drawn in Chapter 4 that the refined RMA-2 is primarily a flood event simulator, rather than a continuous simulation model. This is sufficient for the examination of flood inundation problems being considered here, but may constrain certain future applications. In this event, possible modifications could include enhancement of the wetting/drying routine or implementation of a moving boundary finite element solution, rather than one based on a fixed grid.

5.4 Summary

This Chapter has demonstrated the potential of the refined version of RMA-2 for application to a wide range of floodplain systems. By implementing the developments to modelling strategy outlined in Chapter 3, and theoretically verified in Chapter 4, it has been possible to develop stable initial conditions for floodplain environments of great topographic complexity and implement viable model parameterizations from data that is often inadequate. **Comparisons of model predictions to observed process behaviour at the downstream system outlet have shown a good level of correspondence**, thereby confirming the utility of the refined RMA-2 model for simulation of river channel/floodplain flow at the long reach scale. In addition, a number of alternative modelling strategies and calibration scenarios have been examined.

Having successfully completed this initial evaluation of RMA-2's ability to simulate flow processes in compound meandering channels it is now appropriate to proceed with the next stage in the research design outlined in Chapter 3. This consists of the

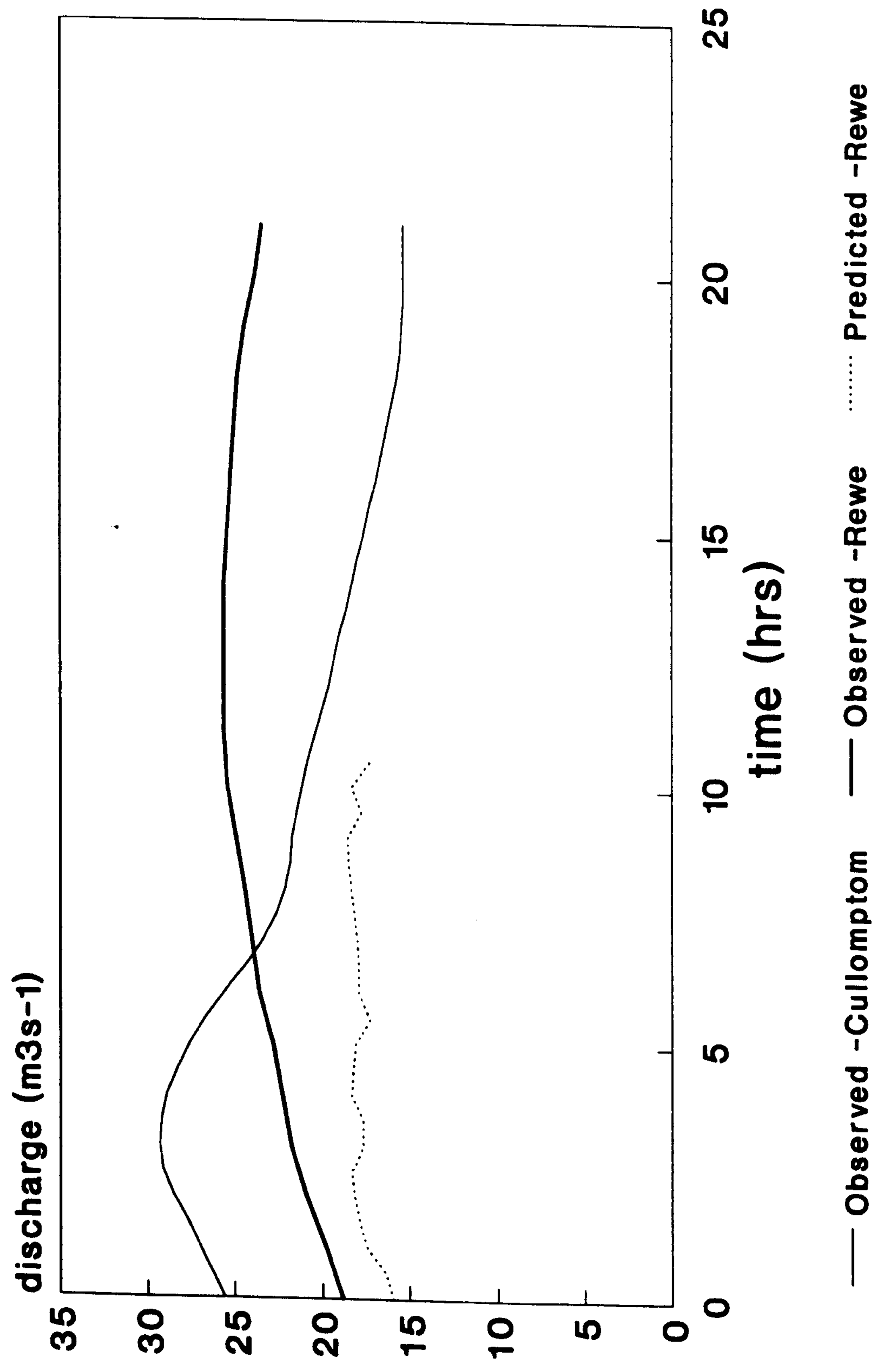


Figure 5.15: RMA-2 simulation of low flow inputs showing observed discharge at Cullompton and observed and predicted discharge at Rewe with $n_{floodplain}$ = 0.045.

further validation of model simulations, but uniquely extends this strategy to cover predictions of 'internal' process behaviour for the entire reach.

CHAPTER 6

Further model evaluation: internal process behaviour

Chapter 5 has described the application of the enhanced RMA-2 model to an 11 km reach of the River Culm, Devon, UK and successfully validated model simulated process behaviour at the downstream system outlet, namely the gauging station at Rewe. In previous modelling studies, model prediction validation has typically limited itself to the above assessment. It is argued here that for complex environmental models this procedure is inadequate and model evaluation methodologies must be constructed that include an examination of 'internal' process behaviour. In particular, this Chapter investigates the ability of the RMA-2 model to simulate dynamic patterns of floodplain inundation over the **entire** reach.

6.1 Introduction

There is no existing physically based modelling scheme capable of high resolution simulation of the spatial and temporal dynamics of inundation phenomena in floodplain environments. Current two dimensional flow modelling schemes either apply to reaches much shorter than those over which floodplain inundation typically occurs or do not consider compound channel cross sections at all. One dimensional schemes, which can be applied at the relevant scale, represent the flow field in terms of water surface elevations at a series of cross sections rather than as a more physically realistic continuous field. Furthermore, one dimensional schemes do not give an indication of lateral flow distributions across the floodplain. The refined RMA-2 modelling scheme outlined in Chapter 3 has the potential to overcome these problems. The research reported in this Chapter therefore explores this potential, with the aim of providing an experimental medium for the exploration of flood inundation processes. Given the difficulty of field data collection in this area and the failure of previous modelling studies, such a development currently represents the only viable tool with which to conduct research essential to future progress in a variety of fields ranging from geomorphology to flood forecasting.

Development of this capability raises a further issue, central to current debate in physically based distributed hydrological modelling (Anderson and Burt, 1990), namely the representation of internal process behaviour. Physically based distributed models in hydrology have typically been evaluated on the basis of predicted output from the catchment or reach length in question (see for example Binley *et al.*, 1991), due to the constraints imposed by data availability. Thus, in a majority of cases, model evaluation has consisted of comparing observed and predicted hydrographs at the modelled system's outlet. Validation of the model for this specific property is used to imply that the representation of catchment processes within the model, such as surface runoff, evapo-transpiration, variations in the phreatic water surface, are a good description of physical reality. However, one of the main advantages of physically based distributed schemes, put forward at their inception, was the ability to generate spatially distributed process predictions for the entire catchment (Freeze and Harlan, 1969; Abbott *et al.*, 1986). Validation of this internal process representation has only been undertaken in a very few studies. Where this has occurred considerable prediction uncertainty has been shown for current model formulations (see for example Anderson and Burt, 1990). Further, it has been demonstrated (Rogers and Anderson, 1987) that identical hydrograph predictions can be generated from a range of process scenarios, which may have varying degrees of validity. Accurate hydrograph predictions may consequently result from process representations that are physically unrealistic for the particular application in question. It is therefore essential that all pertinent aspects of model behaviour are examined to fully validate any complex simulation model.

Lack of attention to the validation of internal process representation in physically based distributed hydrological models represents both a failure to exploit a major potential of this particular modelling strategy and a significant constraint on future developments. Inappropriate model refinements may be attempted due to an uncritical acceptance of current model formulations. Moreover, the opportunity to create models that are better descriptors of internal process behaviour will be lost. In this manner, the future ability of distributed models to make a contribution to both theory and process knowledge will be diminished. Further research is therefore necessary to rigourously assess all aspects of model performance if we are to establish a sound basis from which model development can proceed. It is no longer sufficient for physically based model studies to only implicitly consider internal process representation. To fulfil the needs of research workers and environmental managers, models are required that adequately describe such processes over the entire catchment and thereby reduce the predictive uncertainty associated with currently available schemes. **This will only be achieved with the development of model evaluation methodologies that explicitly assess internal process representation in distributed environmental models. As an initial task, research is required that assesses the potential utility of such an approach.**

This Chapter therefore has two specific objectives:

(i) To describe the assessment of a modelling capability developed for the simulation of spatial and temporal inundation dynamics at a high level of resolution.

(ii) to present an example of a modelling study that uniquely attempts to examine the internal process behaviour of a physically based distributed model and validate these predictions against field observations. By conducting such research it will be possible to draw conclusions concerning the utility of model evaluation methodologies which explicitly consider internal process representation.

6.2 Methodology and experimental design

In the preceding two Chapters evidence has been presented that has successfully validated model predictions against observed field data for the Rewe gauging station. The model is therefore able to reproduce observed process behaviour at the modelled system's downstream external boundary to a satisfactory level of accuracy. In this Chapter, model evaluation is extended to include internal process representation, in particular the prediction of inundation extent. Inundation data is useful in this context as it can be collected at the reach scale (see Section 6.3.1). It is thus possible to use this particular prediction product to validate the **total** internal process environment. All other data relating to internal process representation in models of this type, such as water depths and velocities, are only available for particular cross sections or very localised areas. Inundation extent data therefore represents the primary vehicle for this aspect of model validation. As a minimum requirement the model should be capable of simulating a spatially and temporally dynamic inundation field over horizontal length scales of 10 - 100 m.

Internal process validation will again follow the three stage model evaluation strategy defined by Sargent (1982). Firstly, methods of defining the predicted inundation limit will be identified and proved to be robust. This is analogous to mathematical model validation. Secondly, the computer model will be examined to establish the extent to which it is able to reproduce aspects of the inundation process. This will consist of two phases. As an initial step the sensitivity of inundation predictions to variations in parameter values will be determined. Following this, simulated inundation patterns for two flood events will be examined to determine the model's ability to capture the known spatial and temporal dynamics of the inundation process. The ability of the model to act as a theoretical simulation tool will therefore be demonstrated. Lastly, predictions of maximum inundation extent will be compared to observed field data in order to empirically verify the model.

6.3 Defining flood inundation extent

There is a need to address issues of flood inundation definition in two specific contexts; on the basis of field data and from model results files.

6.3.1 Defining flood inundation extent from field data

Very few data sets of flood inundation extent exist. Those available almost exclusively concern maximum levels of flooding, rather than dynamically changing inundation patterns. The latter requiring synoptic observations of inundation for the entire reach. Inundation data is therefore difficult to collect and is usually reconstructed from historic accounts or *ad hoc* records not specifically taken for the purpose of model validation. For this project two inundation data sets are available for the River Culm study reach. These have been specifically collected for this modelling task and consist of maximum inundation extent data for the two flood events examined in Chapter 5. Inundation observations were taken both during the flood event, by means of air and ground photos, and after, by mapping trash lines on the floodplain which delimit the flow boundary. These data sources have been amalgamated to define the maximum inundation extent for the entire reach for both a 1 in 1 and a 1 in 5 year recurrence interval flood.

6.3.2 Defining flood inundation extent from model data

A basic problem exists with defining flood inundation extent from RMA-2 as the wetting and drying algorithm outlined in Chapter 3 ensures that dewatering elements, that remain within the solution continuum, have a small depth of water maintained across their surface in order to improve model stability. This artificially maintained inundation will therefore lead to errors in model predictions. The solution to this problem lies in applying a mask to the model generated data set which identifies those areas where this artificial water depth exists. These areas may then be eliminated from inundation predictions. As the sensitivity analysis in Chapter 3 has demonstrated that this water depth is < 0.1 m for applications at this scale, a small threshold inundation depth can be set below which all positive water depths predicted by the model are assumed to be an artifact of the wetting and drying algorithm. This assumption obviously results in a loss of detail from the margins of the predicted inundation field (see Figure 6.1). However, this is acceptable given that such loss of detail will only occur for the small number of elements that are specifically dewatering during a given time step and that maximum model parameterization errors generate an inaccuracy in depth prediction that is potentially of a similar order of magnitude. We cannot therefore be sure that predicted inundation within this depth range (0 - 0.1 m) is 'real'. Thus, rather than reducing the model's ability to discriminate areas of inundation, this post processing of model data can be interpreted as helping to remove areas of predictive uncertainty.

It is important, however, to independently establish the validity of the above argument by demonstrating the independence of inundation predictions from the particular masking procedure parameters used. Inundation fields predicted on the basis of the above masking procedure are dependent on two factors; the precise threshold factor used and the minimum domain coefficient, which determines the depth of water maintained by the wetting and drying routine. If variation in these relevant parameters could be shown to have a negligible impact on inundation predictions it

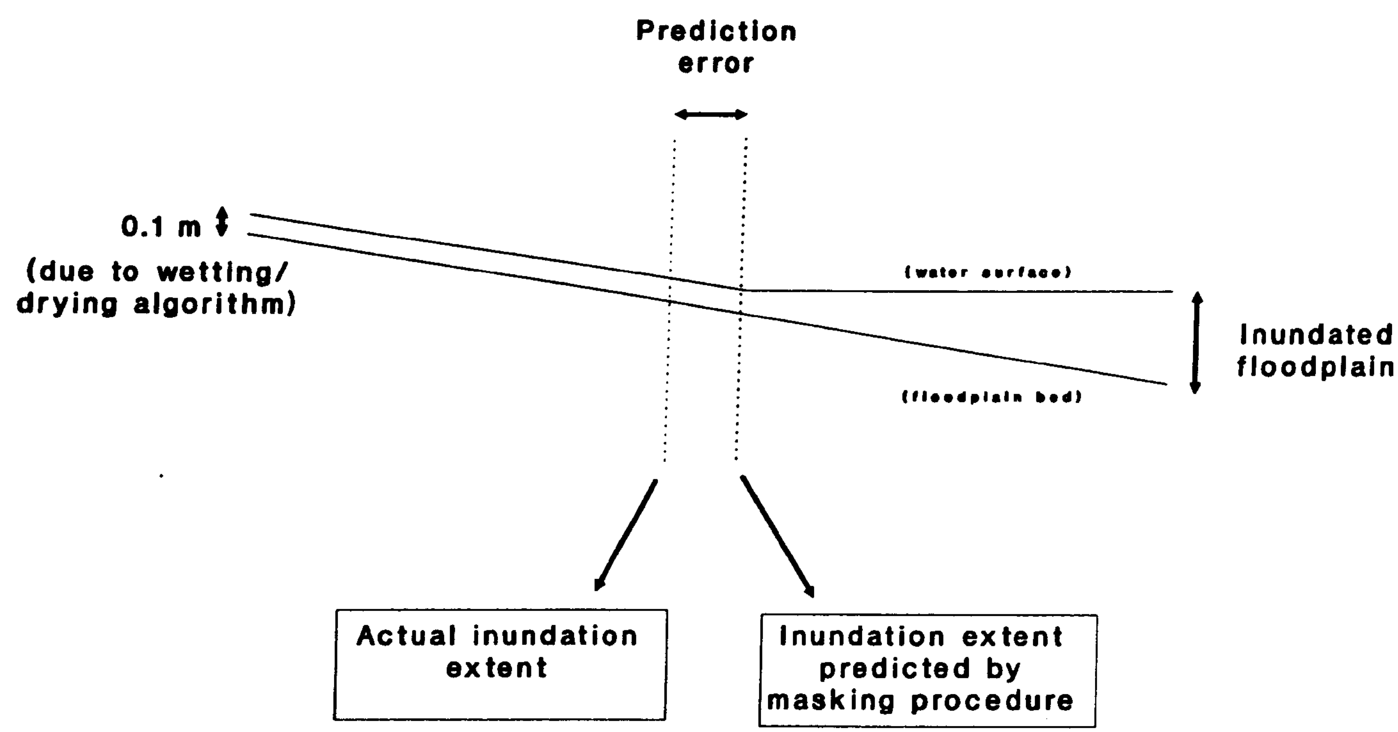


Figure 6.1: Schematic diagram showing potential prediction error caused by the masking procedure used to determine inundation extent from model data.

would be safe to conclude that the masking procedure was robust. To achieve this objective a single factor deterministic sensitivity analysis has been undertaken.

The sensitivity analysis research design implemented to answer the above question assesses the change in predicted inundated area caused by small variations in the normal range of the threshold factor and the minimum domain coefficient. Three values of each parameter were tested for two flood events; the 1 in 1 (event **a**) and the double peaked 1 in 1 and 1 in 5 year (event **b**) recurrence interval events examined in Chapter 5. All other parameter values were held constant. Simulations were then performed using the entire finite element mesh developed for the River Culm study reach (see Chapter 5). This gave a total of 12 dynamic simulations, a task requiring approximately 1 month of continuous real time computing. The percentage inundation change was then calculated for particular time intervals representing rising, peak and falling stages of the discharge hydrograph recorded at Woodmill. This was derived from a contour map of predicted water depths generated from a standard graphics plotting package, UNIRAS. Inundation limits were then digitized and used to derive the areal change. The results of this analysis for the minimum domain coefficient and the masking threshold are shown in Tables 6.1 and 6.2 respectively.

	Minimum domain coefficient		
Flow condition at Woodmill	0.02	0.03	0.04
Rising stage	-	0	1.13
Peak stage	-	0	0.69
Falling stage	-	0	0.90

Table 6.1: Percentage change in inundated area with variation in the minimum domain coefficient. For these calculations control values of 0.02 and 0.13 were used for the minimum domain coefficient and masking threshold parameter respectively.

	Masking threshold (m)		
Flow condition at Woodmill	0.1	0.13	0.16
Rising stage	5.33	-	- 3.68
Peak stage	3.72	-	- 4.1
Falling stage	5.38	-	- 5.33

Table 6.2: Percentage change in inundated area with variation in the masking threshold parameter. For these calculations controls values of 0.02 and 0.13 were used for the minimum domain coefficient and masking threshold parameter respectively

These results indicate that the change in predicted inundated field due to variations in the masking procedure parameters is generally < 5%. This is lower than the overall accuracy of the RMA-2 model for long reach floodplain applications and therefore non-significant. We can thus be confident that model predictions of inundation are not significantly compromised by post-processing. A threshold value of 0.13 m and a minimum domain coefficient of 0.02 were consequently used to establish all inundation predictions presented in this study.

Methods for determining both observed and predicted inundation limits have been presented. These have been shown to be robust relative to the inherent errors in parameter estimation techniques. It is thus possible to proceed with the final two stages of Sargent's three stage model evaluation strategy; a theoretical validation of model simulations and a comparison of model results with observed process behaviour for the Culm catchment.

6.4 Theoretical validation of inundation simulations

Theoretical validation of simulated inundation fields consisted of two components. Firstly, an examination was made of the impact of significant parameters, as determined by the sensitivity analysis presented in Chapter 4, on the predicted inundation field. This was used to determine whether the computer model simulated those inundation processes expected of it, and whether those processes interacted in a logical manner, consistent with the mathematical model. Secondly, spatial and temporal inundation predictions provided by the model were examined to determine the degree to which these are representative of the flood inundation process.

6.4.1 Determining the impact of parameter variation on predicted inundation

The objective of this sensitivity analysis is to determine model response, in terms of inundation, to small variations in the normal range of input parameters.

6.4.1.1 Research design

As in Section 6.3.2, a single factor deterministic sensitivity analysis was selected to achieve the above objective. This methodology is relatively less robust than a stochastic analysis, but is however sufficient to generate all required information, while maintaining reasonable computing requirements. The method used consisted of incrementing each parameter by a small amount while holding all others constant. The percentage change in the predicted inundation field was then established in the manner outlined in Section 6.3.2.

From the preliminary sensitivity analysis on all model parameters reported in Chapter 4, the three most significant (excluding mesh geometry parameters) were selected for further testing here. These were floodplain boundary friction, the domain coefficient depth range and equivalent eddy viscosity. Three values of each of these parameters were selected from the upper, middle and lower parts of their normal field range. Again, dynamic model runs of the 1 in 1 (event **a**) and the double peaked 1 in 1 and 1 in 5 year (event **b**) recurrence interval floods presented in Chapter 5 were then carried out using these parameters. This gave a total of 18 dynamic simulations of the full finite element mesh and required approximately 1.5 months of continuous real time computer simulation to complete. Percentage change in inundated area was then calculated at particular time intervals representing rising, peak and falling stages of the discharge hydrograph recorded at Woodmill.

6.4.1.2 Sensitivity analysis results

The sensitivity analysis results generated by the above research design are shown for events **a** and **b** in Tables 6.3 and 6.4 respectively. As these were generated from unsteady simulations, the response of predicted inundated area to changing parameter values is inherently dynamic. The predicted inundated area therefore both extended and decreased for any particular parameter value. The net change, however, was invariably biased towards one of these directions. Tables 6.3 and 6.4 therefore show this information in both disaggregated and net change formats. The net change for each parameter, amalgamated for both events, are then presented in Figures 6.2 - 6.4. Unit sensitivities have been calculated for this data using Equation 4.1. This has allowed the effect on inundation of the tested parameters to be ranked. The unit sensitivity here refers to the percentage change in inundated area caused by a 100% change in a specific parameter. This data is presented in Table 6.5.

Rank	Parameter	Unit Sensitivity
1	Boundary friction	11.02
2	Domain coefficient depth range	5.77
3	Eddy viscosity	2.07

Table 6.5: Unit sensitivities for model parameters calculated with respect to inundation.

The impact of the above parameter changes in terms of model predicted discharge at Rewe (see Figures 6.5 - 6.8) and volume of water passing this gauging station (see Tables 6.6 and 6.7) has also been examined. In the case of outlet discharge, it can be seen from these results that floodplain boundary friction has most effect on hydrograph shape and is therefore confirmed as the most useful calibration parameter. The other parameters tested appear to have relatively little effect. In terms of event volume, however, the sensitivity to changes in floodplain boundary friction, eddy viscosity and domain coefficient depth range are similar. This indicates that although changes to floodplain boundary friction can bring about alterations to hydrograph shape, volumetric continuity is maintained. These results confirm both theoretical

Flow condition at Woodmill	Increase (%)	Decrease (%)	Net change (%)
Floodplain roughness = - 45.5%			
Rising stage	3.15	0.09	- 3.24
Peak flow	10.1	0.12	- 9.98
Falling stage	6.38	0.12	- 6.26
Floodplain roughness = + 82%			
Rising stage	0.2	5.0	4.8
Peak flow	0.99	2.41	1.42
Falling stage	0	19.3	19.3
Eddy viscosity coefficient = - 90%			
Rising stage	0.53	1.97	1.44
Peak flow	1.96	0.21	- 1.75
Falling stage	2.68	0.12	- 2.56
Eddy viscosity coefficient = + 100%			
Rising stage	0.15	4.17	4.02
Peak flow	0.23	1.61	1.38
Falling stage	1.08	2.91	1.83
Domain coefficient depth range = - 16.7%			
Rising stage	0.29	1.08	0.79
Peak flow	2.21	0.24	- 1.97
Falling stage	1.29	0.3	- 0.99
Domain coefficient depth range = + 33.3%			
Rising stage	0.11	2.97	2.86
Peak flow	0.27	2.54	2.27
Falling stage	0.69	0.69	0

Table 6.3: Changes in predicted inundated area for event a caused by variation in three significant model parameters. Control values for these simulations were taken from the parameter set outlined in Chapter 5.

Floodplain roughness = - 45.5%			
Flow condition at Woodmill	Increase (%)	Decrease (%)	Net change (%)
Rising stage	9.36	3.19	- 6.17
Peak flow	1.59	0	- 1.59
Falling stage	11.21	0	- 11.21
Floodplain roughness = + 82%			
Rising stage	0	14.55	14.55
Peak flow	0	0.78	0.78
Falling stage	0	9.97	9.97
Eddy viscosity coefficient = - 90%			
Rising stage	2.24	0.19	- 2.05
Peak flow	0.38	0.01	- 0.37
Falling stage	1.7	0.38	- 1.32
Eddy viscosity coefficient = + 100%			
Rising stage	0	4.67	4.67
Peak flow	0.16	0.14	- 0.02
Falling stage	0.05	4.28	4.23
Domain coefficient depth range = - 16.7%			
Rising stage	2.44	0.01	- 2.43
Peak flow	0.58	0.08	- 0.5
Falling stage	1.23	0.17	- 1.05
Domain coefficient depth range = + 33.3%			
Rising stage	0.14	3.14	3.0
Peak flow	0.09	0.17	0.08
Falling stage	0.01	2.24	2.23

Table 6.4: Changes in predicted inundated area for event **b** caused by variation in three significant model parameters. Control values for these simulations were taken from the parameter set outlined in Chapter 5.

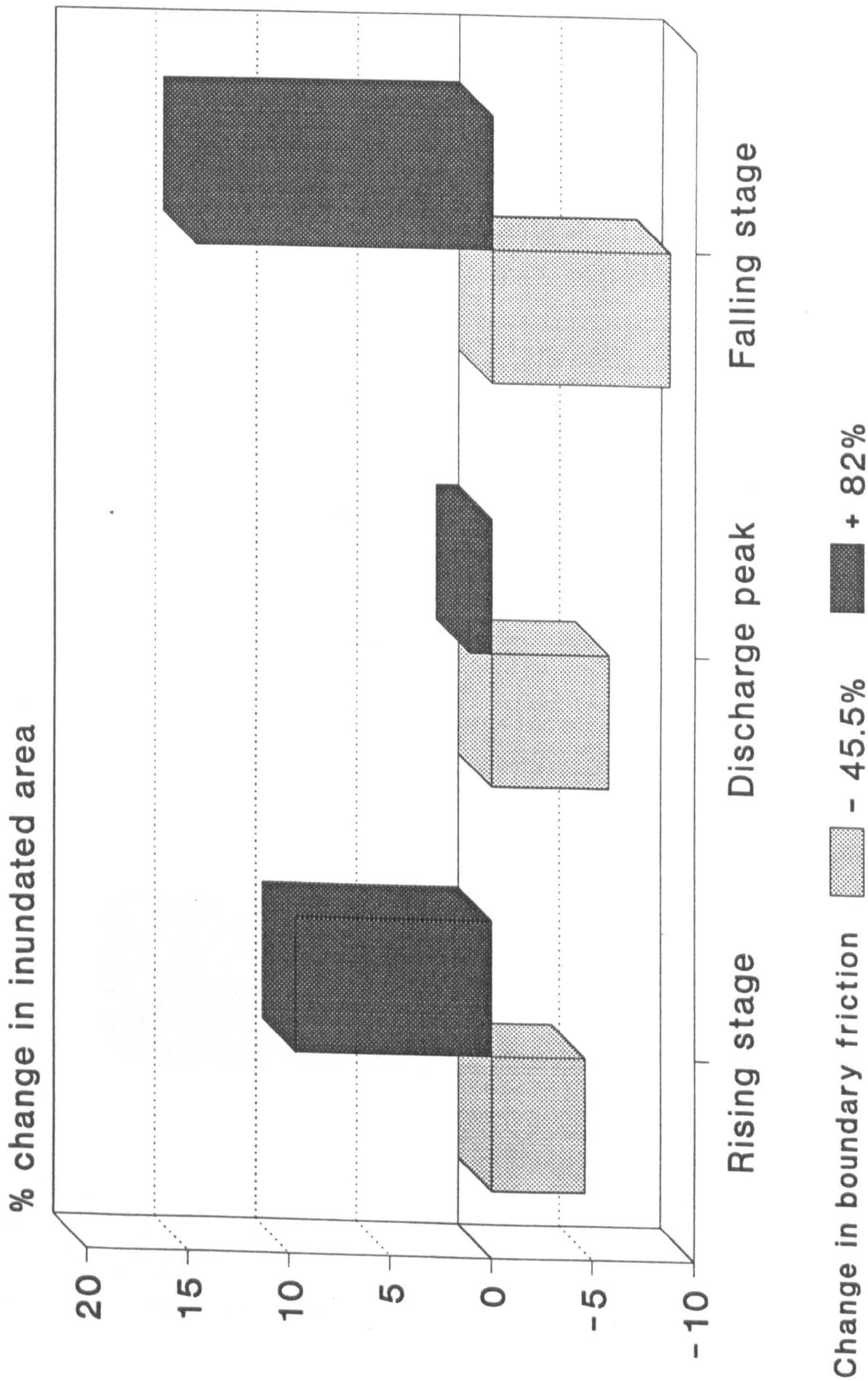


Figure 6.2: The impact of variation in floodplain boundary friction on predicted inundated area.

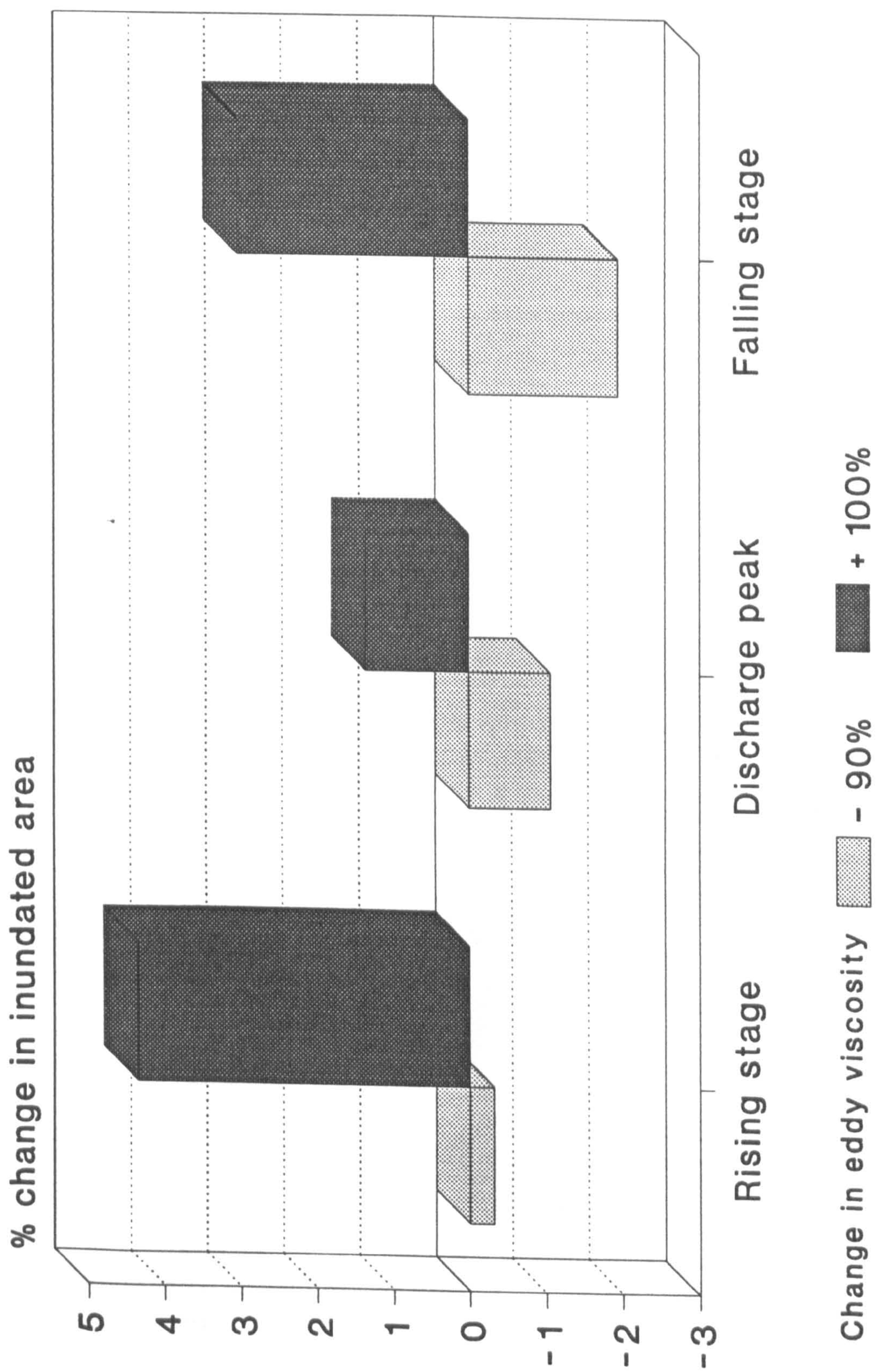


Figure 6.3: The impact of variation in eddy viscosity on predicted inundated area.

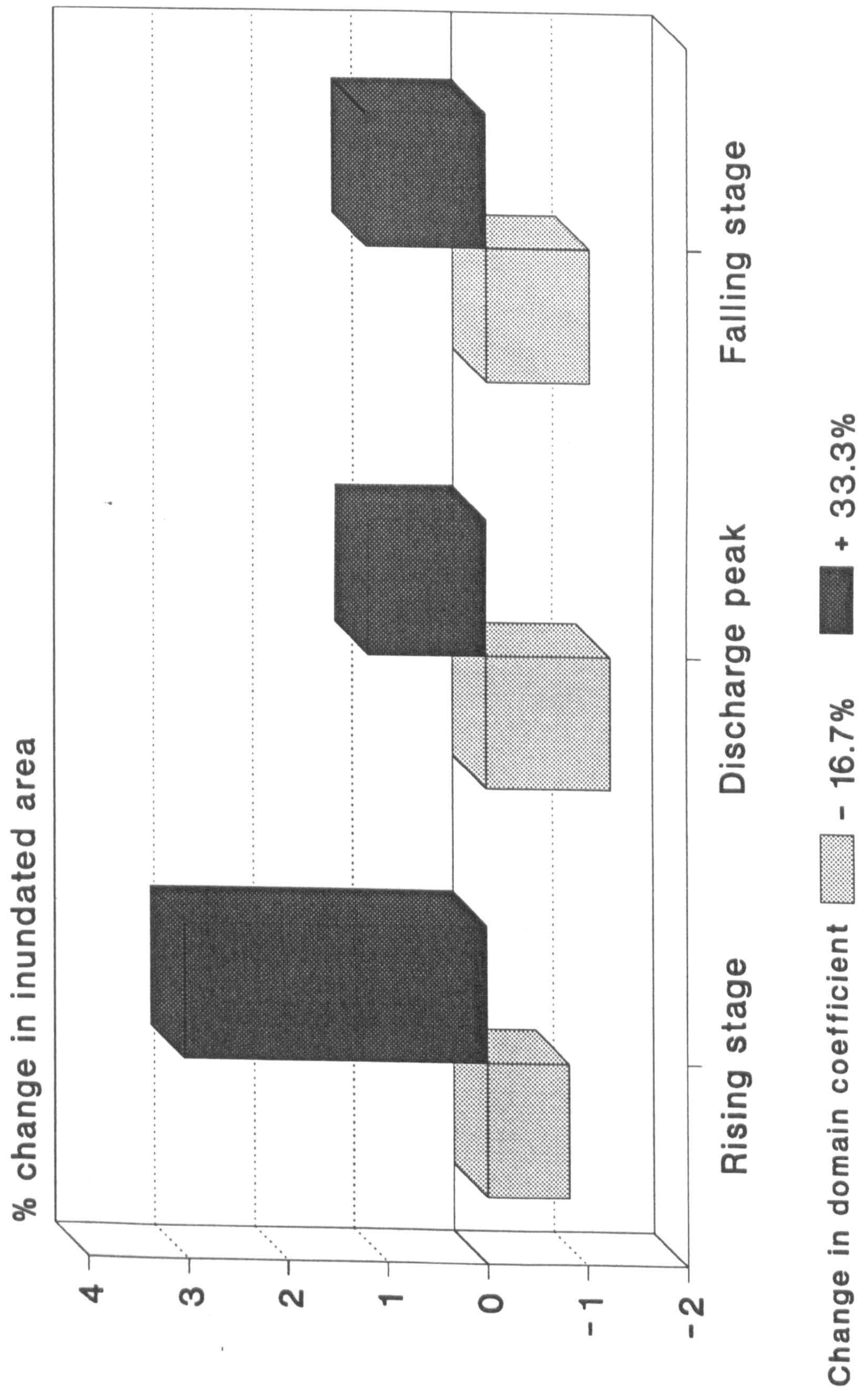
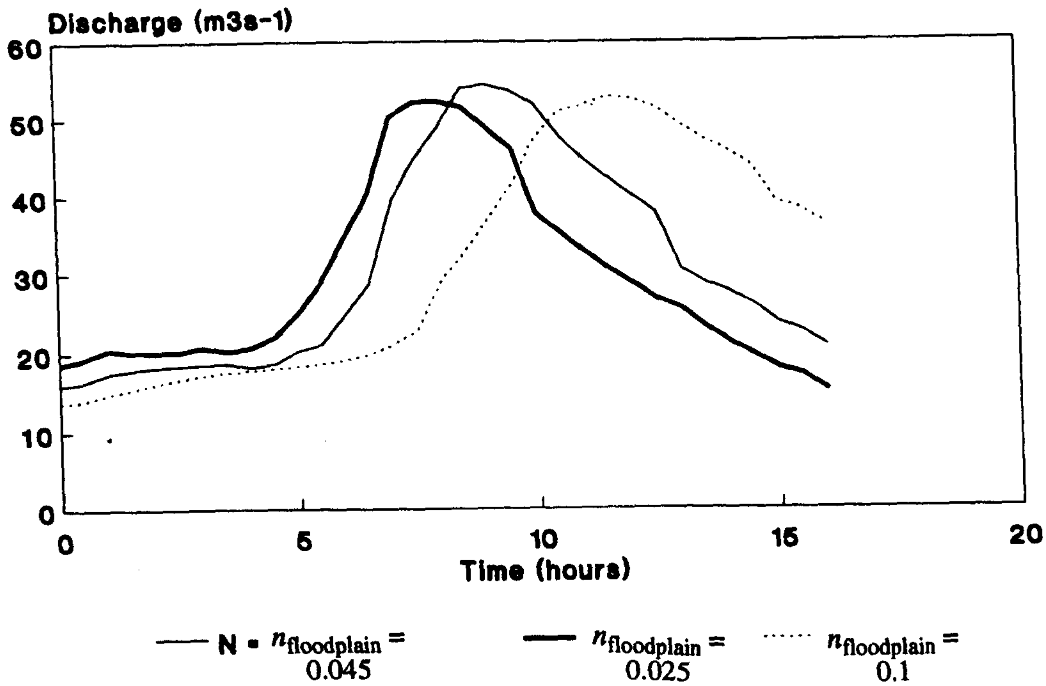


Figure 6.4: The impact of variation in domain coefficient depth range on predicted inundated area.

(a)



(b)

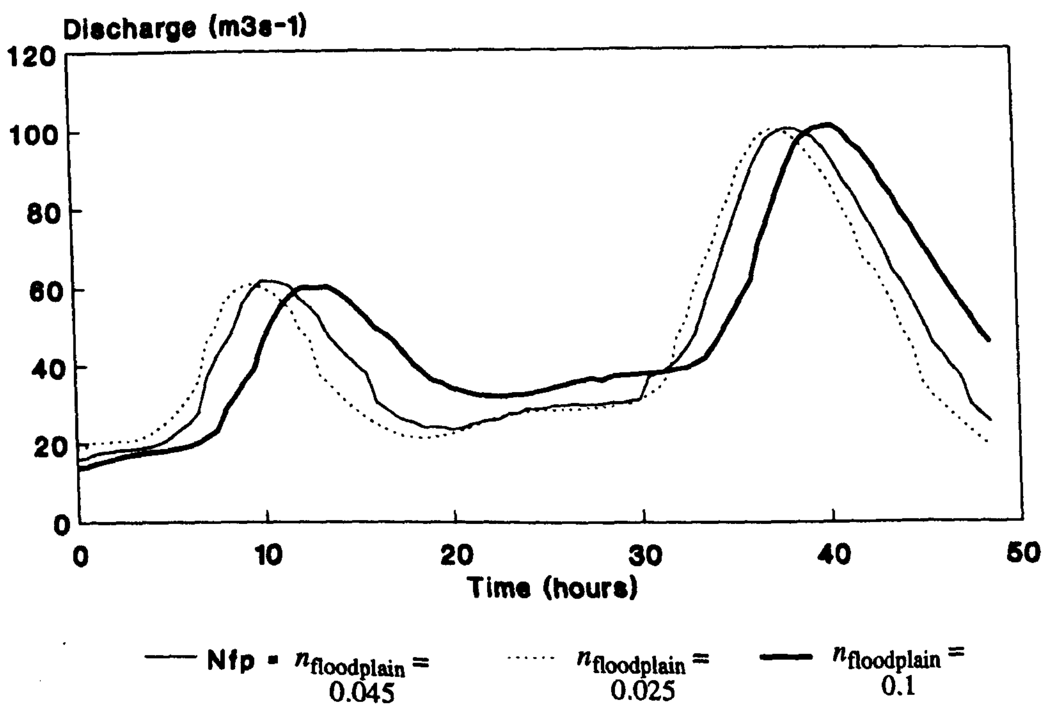
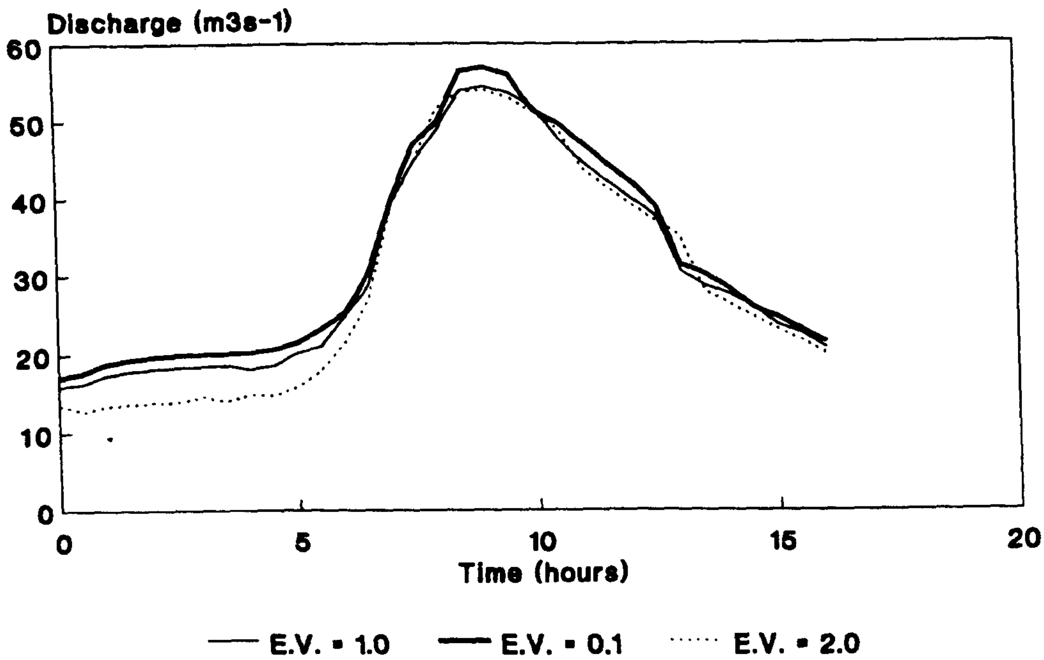


Figure 6.5: The impact of variation in floodplain boundary friction on model predicted discharge at Rewe for a 1 in 1 year recurrence interval event (a) and a double peaked event (b), consisting of 1 in 1 and 1 in 5 year recurrence interval events.

(a)



(b)

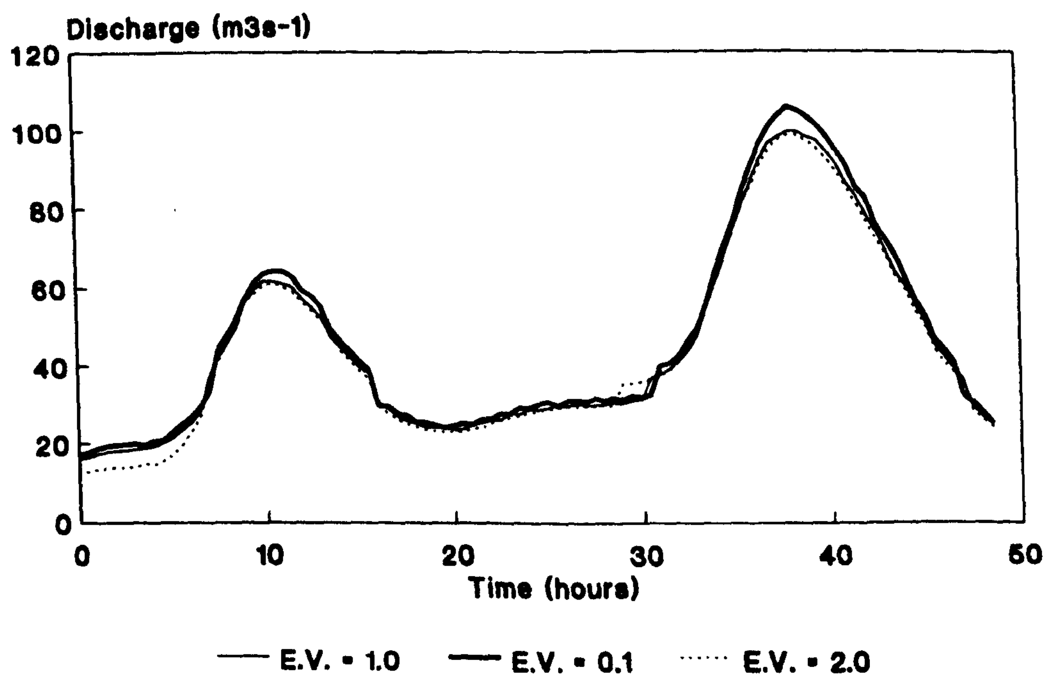


Figure 6.6: The impact of variation in eddy viscosity on model predicted discharge at Rewe for a 1 in 1 year recurrence interval event (a) and a double peaked event (b), consisting of 1 in 1 and 1 in 5 year recurrence interval events.

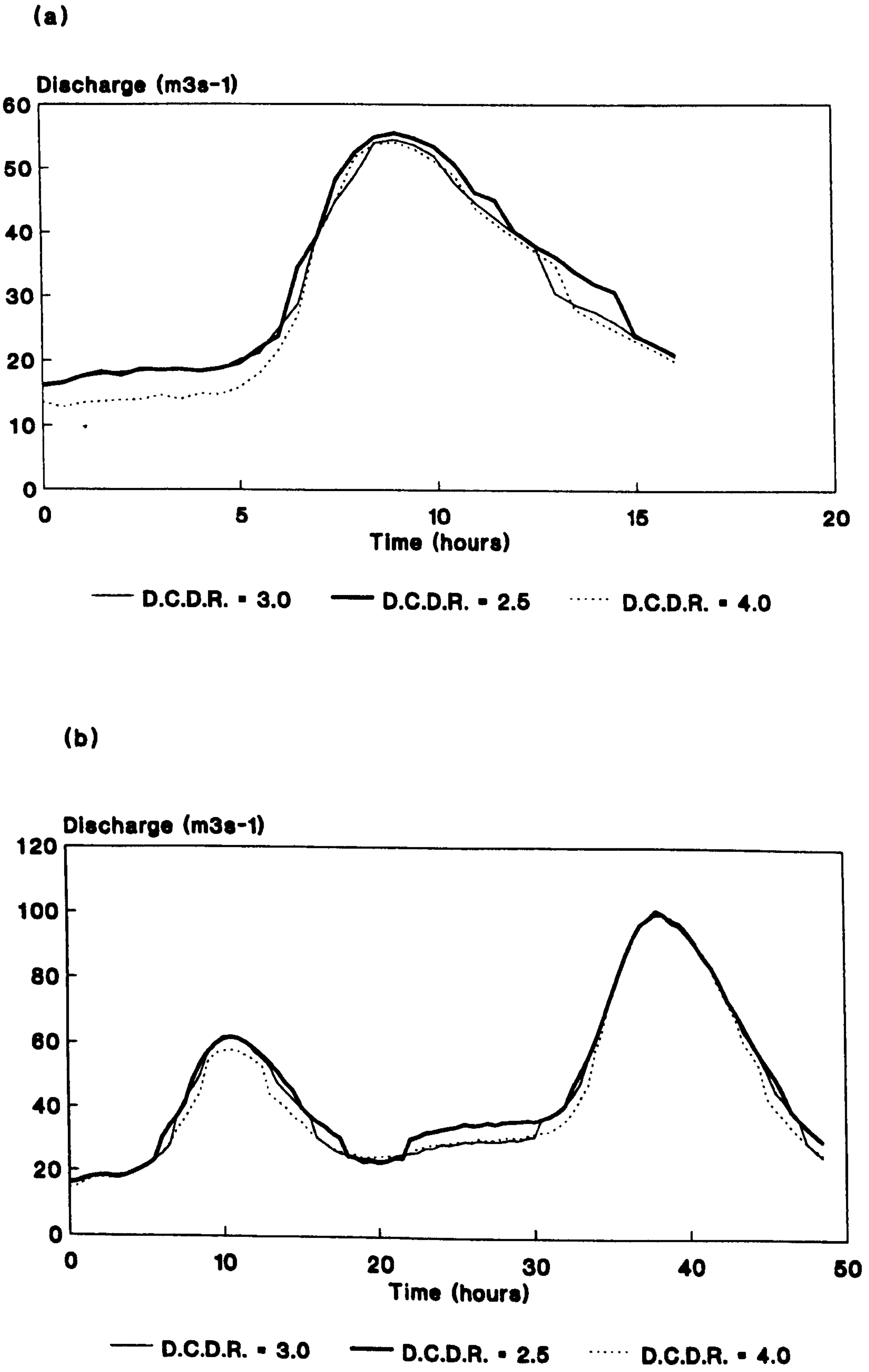
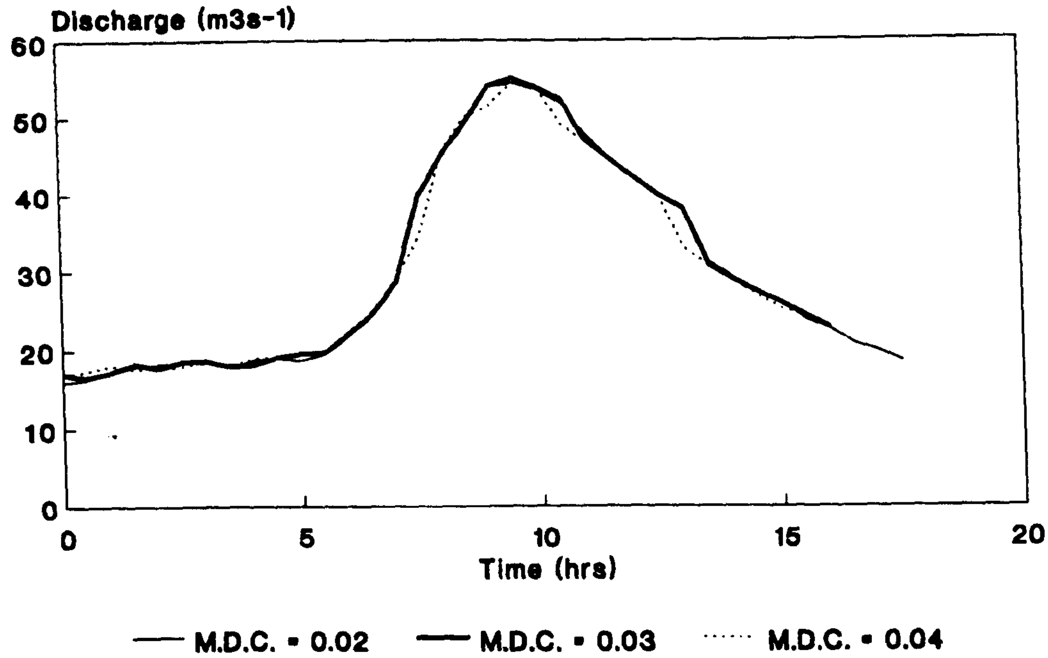


Figure 6.7: The impact of variation in domain coefficient depth range on model predicted discharge at Rewe for a 1 in 1 year recurrence interval event (a) and a double peaked event (b), consisting of 1 in 1 and 1 in 5 year recurrence interval events.

(a)



(b)

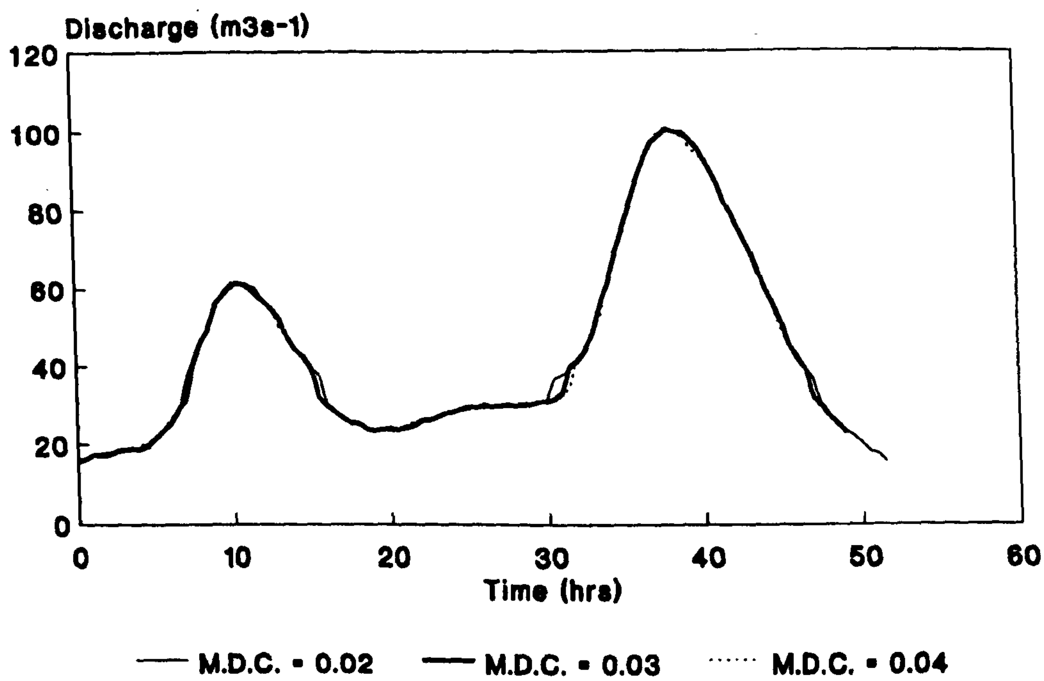


Figure 6.8: The impact of variation in minimum domain coefficient on model predicted discharge at Rewe for a 1 in 1 year recurrence interval event (a) and a double peaked event (b), consisting of 1 in 1 and 1 in 5 year recurrence interval events.

Simulation	Volume (m^3 x 10^6)	% Change
Control	1.81923	n.a.
$n_{floodplain}$ = 0.025	1.72954	- 4.93
$n_{floodplain}$ = 0.1	1.8466	1.5
E.V. = 0.1	1.904	4.66
E.V. = 2.0	1.72274	- 5.3
D.C.D.R. = 2.5	1.90036	4.46
D.C.D.R. = 4.0	1.68663	- 7.29
M.D.C. = 0.03	1.82727	0.44
M.D.C. = 0.04	1.8001	- 1.05

Table 6.6: Changes in event volume due to parameter variation; event **a**.

Simulation	Volume (m^3 x 10^6)	% Change
Control	7.91906	n.a.
$n_{floodplain}$ = 0.025	7.59298	- 4.12
$n_{floodplain}$ = 0.1	8.39944	6.07
E.V. = 0.1	8.22528	3.87
E.V. = 2.0	7.7457	- 2.19
D.C.D.R. = 2.5	8.20905	3.66
D.C.D.R. = 4.0	7.63596	- 3.57
M.D.C. = 0.03	7.87327	- 0.56
M.D.C. = 0.04	7.85405	- 0.82

Table 6.7: Changes in event volume due to parameter variation; event **b**.

expectations and the results of earlier analyses (see Chapter 4). In terms of calibration therefore, eddy viscosity and the domain coefficient depth range can be most usefully employed to 'fine tune' model simulations, should this be considered necessary. Potentially, they may also have the ability to alter velocity distributions across the channel/floodplain cross section, although this particular aspect of model behaviour will require further field data collection programmes before meaningful evaluation can be undertaken.

In relation to the results of this analysis a number of points can be made. Firstly, the magnitude and sign of net inundation change caused by parameter variation is consistent with expectations. The model therefore represents those inundation processes expected of it in a logical manner, consistent with the mathematical model. Secondly, the model's response to parameter variation is spatially dynamic. This attribute of the physical system is implied from a consideration of other flow phenomena in compound meandering channels, such as momentum transfer, but has not been previously demonstrated for the inundation process. Thirdly, the model's sensitivity to inundation appears to be temporally variable. Figures 6.2 - 6.4 demonstrate significantly smaller changes in inundation with parameter variation under peak flow conditions than at other times. However, as the change in inundation extent is calculated as a percentage of the total floodplain area, it is likely that the computational procedure accounts for a substantial component of this effect. Lastly, the ranking of parameters is broadly consistent with the first pass sensitivity analysis of all parameters undertaken in Chapter 4. Moreover, this provides confirmation that the conclusions drawn in Chapter 4 apply to full size meshes as well as subsections. These analyses vary, in that boundary friction has supplanted the domain coefficient depth range as the most sensitive parameter. This is however a minor discrepancy, likely to be due to a difference in the scale of mesh and the particular simulation events used.

6.4.1.3 Summary

Model response, in terms of predicted inundation extent, has been determined for the schemes most significant parameters. This has allowed confirmation to be made of the model's ability to represent the inundation process in a dynamic manner consistent with the mathematical model and known process behaviour (see Section 2.1.3).

6.4.2 Determination of the model's ability to simulate dynamic inundation fields

Inundation time sequences, predicted by the model, for each of the above events have been determined from simulations carried out using the parameter set outlined in Chapter 5, with the inundation extent determined by the procedure outlined in Section 6.3.2. For this initial analysis inundation extent was calculated at three hour time intervals. This was felt to be an adequate compromise between demonstrating the major characteristics of the simulated inundation field and the need to keep computing requirements reasonable. Increasing temporal resolution at this stage of the investigation would not be justified in terms of the limited additional information that this would generate, despite a significant increase in processing time. These time sequences were then examined to determine the model's ability to simulate a spatially and temporally dynamic inundation field. The percentage inundated area at each time interval for both events has been calculated and is presented in Figure 6.9a and 6.9b. Plots of inundated area for two time intervals (3 and 9 hours) for the 1 in 1 year recurrence interval event (event **a**) are presented in Figure 6.10.

These results demonstrate the ability of the model to simulate the passage of a flood wave along reaches approximately an order of magnitude longer than those previously considered by this class of model. Even complicated flow conditions, such as the double peaked flood event (event **b**), are simulated in a physically realistic fashion. In this case, a second flood wave arrives in the reach while the floodplain is still dewatering from a previous inundation episode. This later event then causes further substantial inundation. The model therefore has the capability to dynamically simulate spillage of water onto the floodplain, the lateral extension of this inundation front and its subsequent return to the channel. This is consistent with current knowledge concerning process behaviour for a large number of floodplain environments.

Figure 6.9 highlights the point that the concept of a maximum inundated area for a particular magnitude event is, in reality, physically unrealistic as it is based on a static, rather than a dynamic, view of the inundation process. The maximum inundated area represents the cumulative inundation pattern caused by a floodwave passing through a reach. The actual maximum inundated area that can physically occur during the course of any single event is therefore smaller than this value. Investigations into floodplain system phenomena which depend on dynamic effects, such as sediment deposition problems, must take this into account. In addition, the

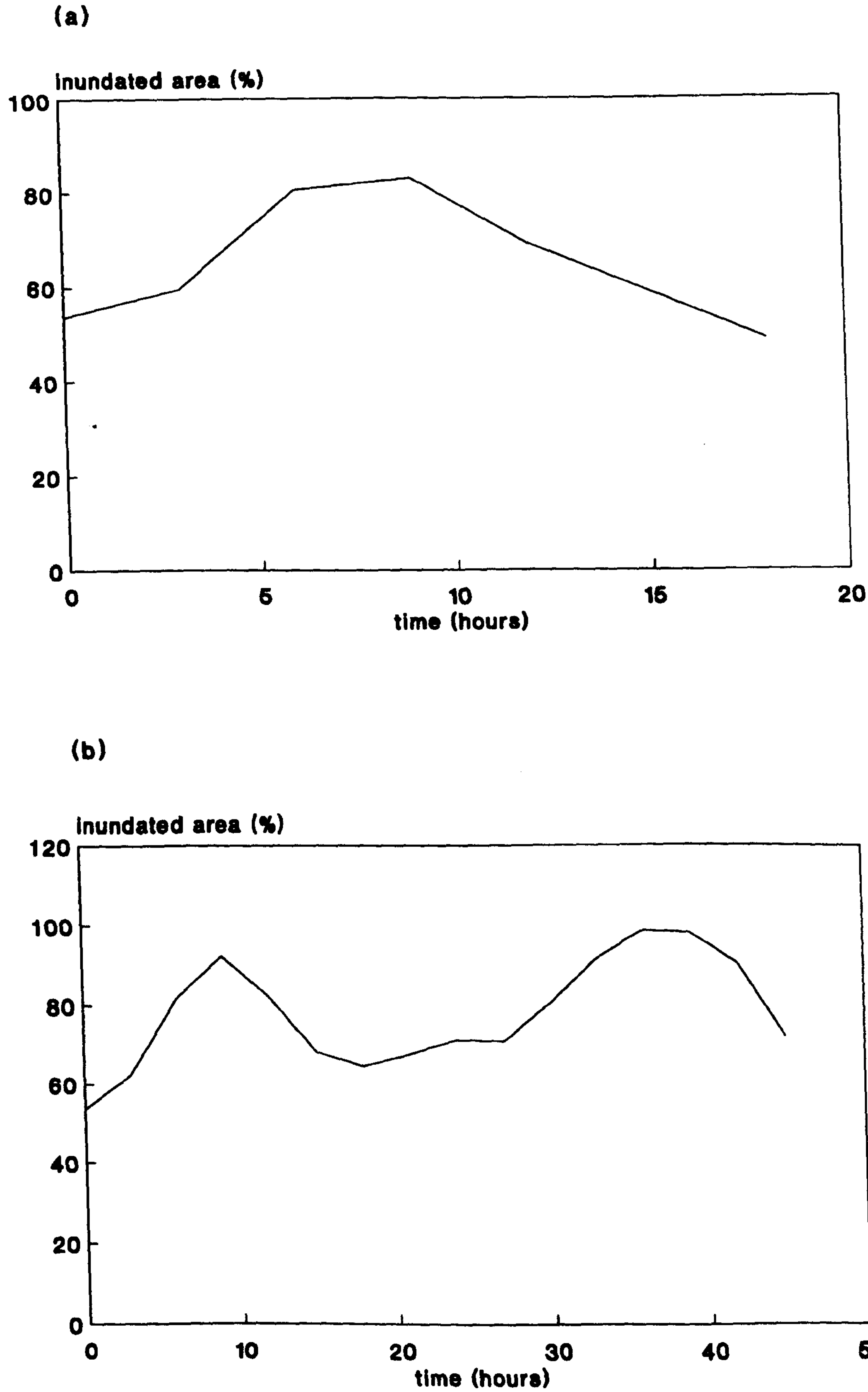


Figure 6.9: Model predicted time sequences of inundated area for (a) a 1 in 1 year event and (b) a double peaked event consisting of 1 in 1 and 1 in 5 year events.

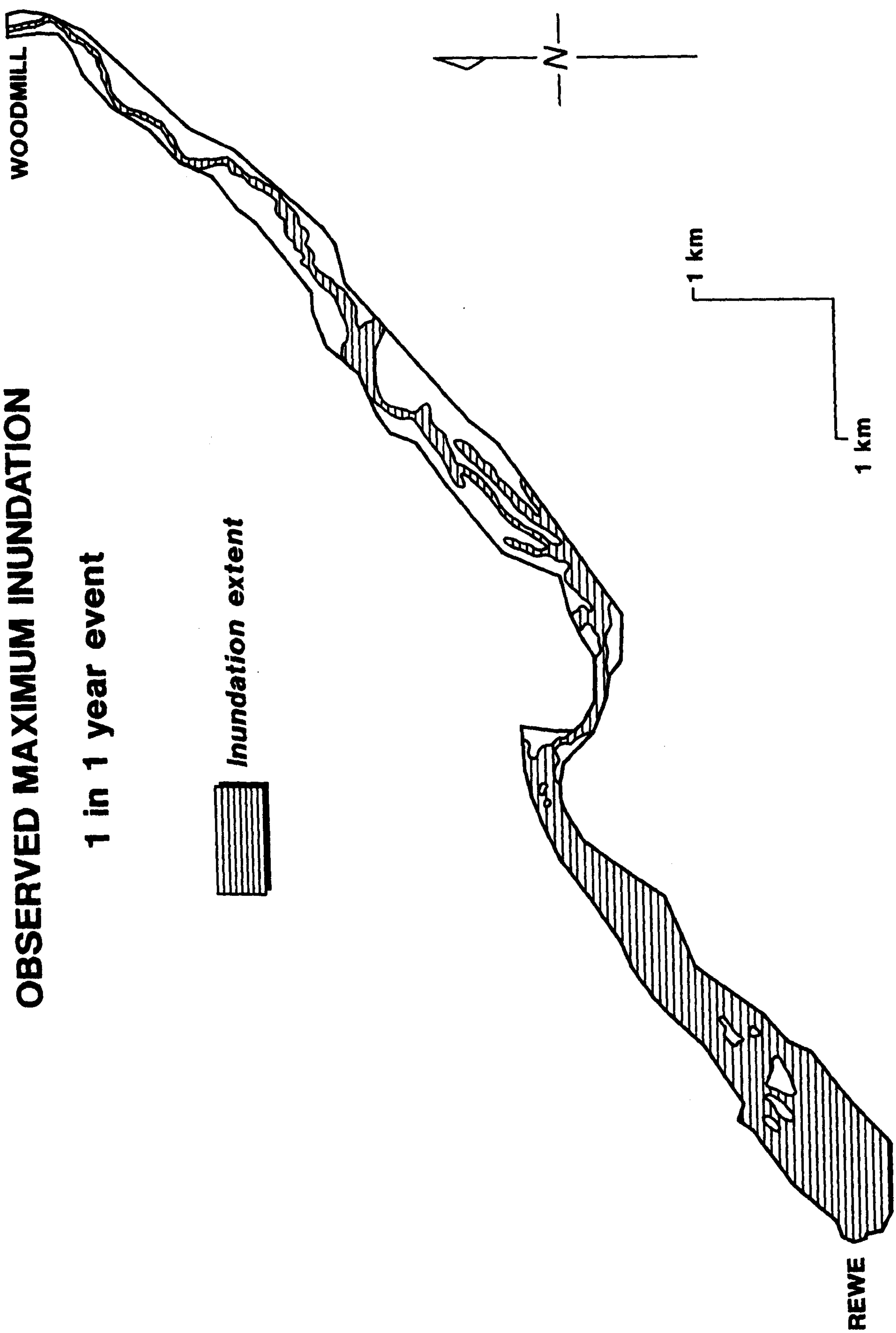


Figure 6.9c: Observed maximum inundation for a 1 in 1 year recurrence interval flood (event a).

two 1 in 1 year recurrence interval events presented in Figure 6.9a and 6.9b show quantitative differences in terms of their inundation time sequences. The percentage inundated area attained by the 1 in 1 year event forming part of event **b** is significantly higher than that for event **a**. The model is therefore able to discriminate differences in inundation patterns for particular events. Such differences cannot at present be confirmed by field data, however in the light of the complex hydraulics of river channel/floodplain systems it is highly likely that such effects exist. Given this, the development of a modelling capability that can undertake this type of simulation is a major advance on previous model formulations. Thus, for the first time we are able to develop model scenarios to investigate inundation dynamics at an appropriate scale. Observed maxiumum inundation for event **a** is shown in figure 5.9c.

Figure 6.10 clearly shows the ability of the model to simulate inundation at a high level of spatial resolution. Flow patterns with horizontal length scales as low as 10 m can be delimited, as can the impact of larger topographic features, such as the railway embankment (feature 2 on Figure 5.4). For the 9 hour time interval on Figure 6.10, a pond of water has formed upstream of this feature in contrast to the dry region of floodplain downstream. The embankment therefore causes significant backing up of flow. As a further example, a complex floodplain island can be seen to form within the arms of the channel bifurcation (feature 4 on Figure 5.4). This is progressively swamped at higher flows. The model is therefore able to simulate the spatial and temporal dynamics of floodplain inundation fields at high levels of resolution.

A more general modelling problem highlighted by a move towards the evaluation of internal process behaviour for complex environmental models is caused by the large quantity of data generated. Typical simulations, such as those reported in this Chapter involve the derivation of data sets consisting of tens of thousands of data items, located within a three dimensional spatial matrix, for each time step. Recourse is therefore made to arbitrary time slicing of such data sets in order to reduce problems of data handling and visualisation. Such a procedure has proved acceptable for the first pass analysis of internal process behaviour presented here but will, inevitably, be inadequate for further detailed study. A need has therefore been demonstrated to develop techniques that allow the analysis of the total data set generated by large environmental models. This is essential to the further exploration of the model evaluation methodologies of the type presented in this Chapter. One potential technique is the use of video post processing to display time series of

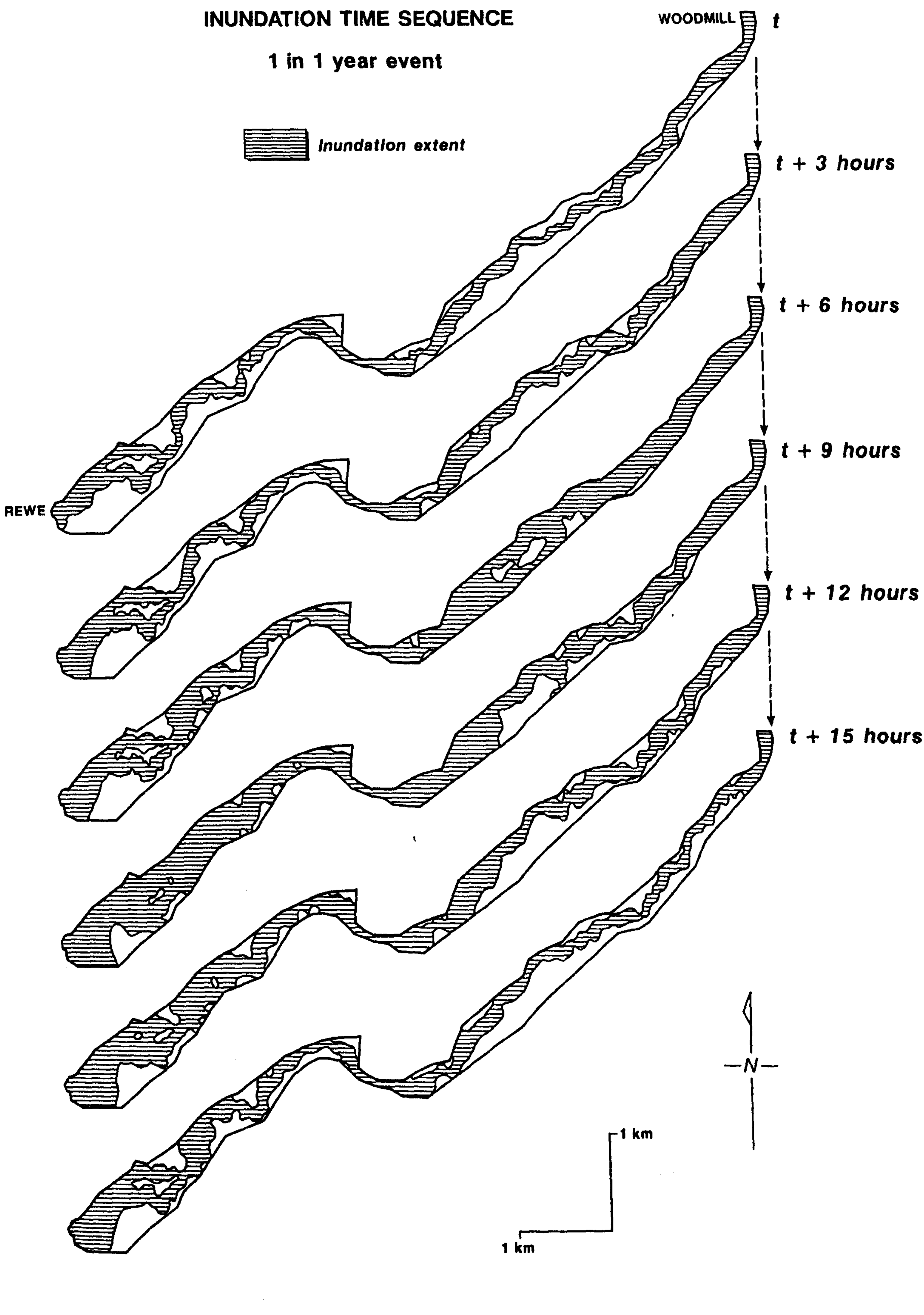


Figure 6.10: Model predicted inundation extent at 3, 6, 9, 12 and 15 hours of a simulated 1 in 1 year recurrence interval flood (event a).

computer graphics. This would allow model evaluation hypothesis relating to internal process behaviour to be more rigourously falsified.

6.5 Validating predicted inundation against field data

Having established that model simulated inundation fields are theoretically correct, being consistent with the mathematical model and current understanding of the physical system, it is possible to proceed with a comparison of these predictions with field data. As indicated in Section 6.3.1 maximum inundation extent data has been collected for both flood events analysed in Section 6.4. This has been compared to the maximum inundation extent predicted by the model (Figure 6.11). This comparison demonstrates the ability of the model to explain approximately 60 - 90% of the variation in the maximum inundation data set, with errors tending to under- rather than overpredict. Moreover, the recurrence frequency at which total inundation of the floodplain occurs is also accurately predicted by the model, as these floods broadly cover the entire range of inundation event magnitudes, from the just out-of-bank condition to complete floodplain inundation. It is, however, important to note that predicted and observed inundation fields will inevitably converge as 100% floodplain coverage is approached. This may explain the difference in prediction accuracy between events **a** and **b**. As a minimum position we are therefore able to conclude that the accuracy of predicted inundation fields derived from the RMA-2 model is in excess of 60%. However, as only two data sets, both relating to flood inundation, have been used in this evaluation process, to fully confirm these findings it would clearly be of benefit to attempt further validation of internal process predictions against a third independent data set.

The effect of constriction by the downstream control structure is also easily observable in Figure 6.11. Flow can be seen to locally back up at the downstream control structure as flow depth increases during flood events. A further anomaly in the inundation data also occurs in the upper 1 - 1.5 km of the finite element mesh. The model predicts total inundation of the floodplain in this region at relatively low flows. This is in direct contrast to the field data, which indicates in-bank flow only. This effect appears to be caused by the impact of the upstream control structure on model results. The purpose of the control structure is to ensure the direction of flow entering the mesh is parallel with the sides, in order to avoid the generation of physically unrealistic flow fields. In this case the control structure appears to retard

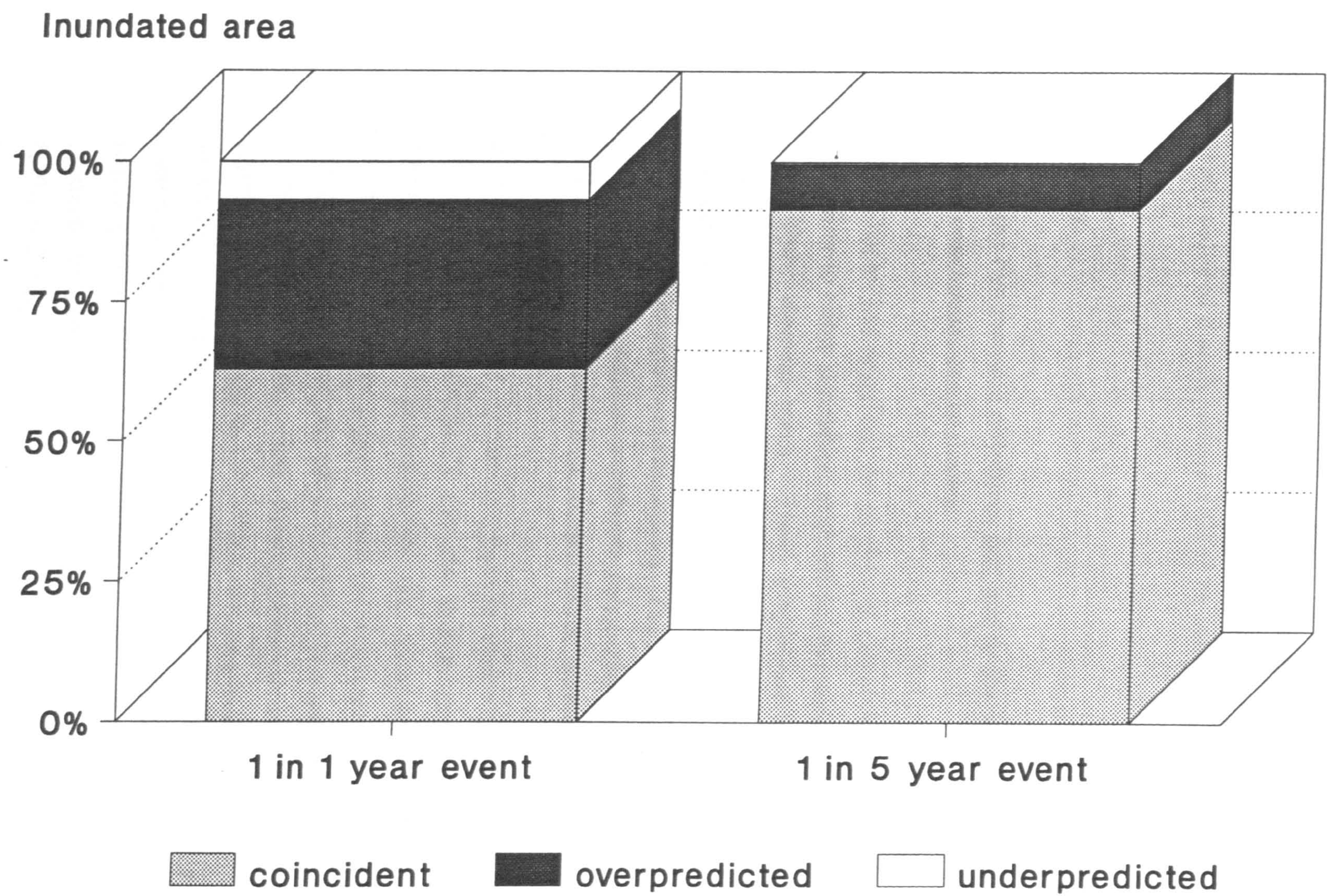


Figure 6.11: A comparison of model predicted inundation patterns to field data taken for two specific events; a 1 in 1 year recurrence interval flood (event a) and a double peaked flood (event b) consisting of 1 in 1 and 1 in 5 year recurrence interval events.

the velocity of water entering the mesh. Water depths are therefore overpredicted. It is clear that for this class of model the order of magnitude increase in scale of application implemented in this study invalidates previously accepted operational rules concerning the design of such control structures. Further work in this area is consequently required if such problems are to be overcome.

6.6 Summary

In respect of the two research objectives outlined in the introduction to this Chapter the following conclusions can be drawn:

i) This Chapter has described the successful extension of the RMA-2 modelling capability to allow simulation of dynamic inundation time sequences over the entire finite element mesh. These predictions have shown a satisfactory correspondence to field data. This unique modelling capability has good application potential in relation to a number of research and management problems in floodplain environments. For example, information concerning the location of floodplain inundation zones during the course of flood events will allow zones of floodplain accretion to be defined.

ii) Significant potential exists in physical hydrology for model evaluation methodologies that explicitly consider internal process representation. The successful example presented in this Chapter demonstrates that this methodology has the ability to aid the development of models that more completely represent those physical processes occurring within the catchment or reach in question. The potential that a model can falsify physical representation, in order to ensure an accurate predictive capability at specific locations, is thereby reduced. This may result in the rejection of a number of currently used modelling schemes. Future model formulations will be required to produce a greater variety of spatially distributed prediction products to increased levels of tolerance. This requirement will only be fulfilled if model evaluation methodologies, such as the one presented in this Chapter, are adopted.

iii) Although validation of inundation predictions has successfully been confirmed for two test events, to be fully confident of the model's ability to generate accurate internal process representation further validation against a second, independent, data set is required.

The satisfactory completion of this initial phase of internal model prediction evaluation also serves to complete the research design outlined in Chapter 3. It is therefore appropriate at this point to assess the progress made and draw conclusions. This is undertaken in Chapter 7 and will enable potential extensions to the current modelling capability to be suggested. These are then considered in detail in Chapter 8.

III Summary and model extension

CHAPTER 7

Summary of two dimensional finite element modelling capabilities

This thesis has described the further development of a prototype two dimensional finite element model to enable the high resolution simulation of river channel/floodplain flow at reach lengths appropriate to floodplain inundation. Evaluation of this modelling scheme has been undertaken and a test application made to the River Culm, Devon, UK. It is now appropriate to review the findings of this research and determine whether the modelling objectives stated in Chapter 1 of this thesis have been realised. This process will allow a programme of further development, consisting of enhancements to the numerical model and an exploration of potential applications, to be suggested in Chapter 8. Extension of the modelling capability via this latter method is then pursued in this final Chapter on the basis of two application case studies. Chapter 7 therefore provides the necessary linkage integrating these two phases of model development.

7.1 Summary of major research findings

Figure 7.1 delimits the specific advances made by this project. These will now be discussed in detail.

7.1.1 Further enhancement of the RMA-2 modelling scheme

This project has undertaken essential further enhancement of the RMA-2 modelling scheme to enable long reach floodplain flow problems to be simulated at a high level of resolution. This work has consisted of refinement of both the numerical model and the physical representation by finite elements. In particular, modifications to the

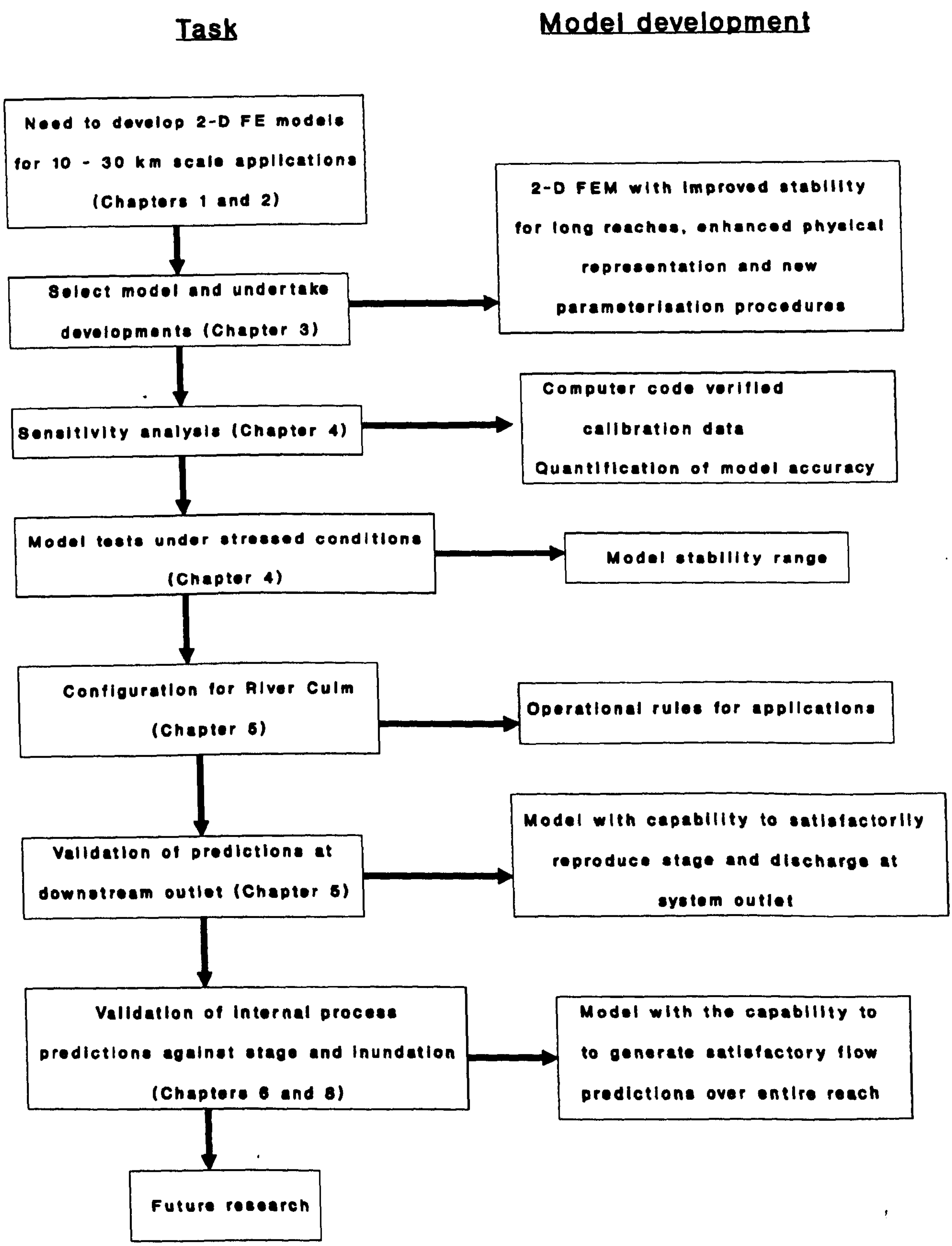


Figure 7.1: Summary of major research advances.

physical model have included alteration of the wetting and drying algorithm to improve stability at long reach scales and the identification of new procedures for determining model boundary conditions in the specific case of compound channels for which adequate flow data is unavailable. Enhancement of the finite element mesh discretization has also been undertaken to enable issues relating to the realistic physical representation of flows over complex topography to be adequately resolved for such models. This has been achieved with a minimum of topographic and flow data input. The resulting scheme is therefore unique in relation to the prototype RMA-2 model, and to two dimensional finite element models in general, in its ability to:

i) represent highly dynamic inundation processes at the long reach scale while maintaining numerical stability,

ii) be parameterized on the basis of extremely sparse data sets,

iii) simulate a channel meandering within a larger floodplain corridor,

iv) include topographic complexity into the finite element discretization at long reach scales,

v) represent river reaches approximately an order of magnitude greater than other two dimensional finite element models.

7.1.2 Model accuracy and stability

A sensitivity analysis for the enhanced RMA-2 model was undertaken on two test reaches to assess model response to the normal range of parameter variation. This has allowed an initial estimate of model accuracy for long reach floodplain applications to be made. Model accuracy was found to be most sensitive to inherent errors in the estimation procedure used to determine the downstream boundary condition. Maximum potential model error was therefore found to vary between ±9.77% and ±23.15% for predicted velocity and between ±6.45% and ±17.54% for predicted depth, depending on the particular estimation procedure used. The lower end of this error range compares favourably with the ±8% error associated with two dimensional depth averaged codes applied under ideal conditions (Norton *et al.*, 1973; Hervout, 1989).

An attempt has also been made to determine the stability range of the enhanced scheme. Continuity and convergence of the numerical scheme was found to be satisfactory for all sensitivity analysis simulations. A further series of model simulations under stressed parameter conditions was subsequently performed to determine where this stability broke down. It was concluded that numerical stability problems did not compromise the model's ability to fulfil the objectives set out in Chapter 1, although an instability at low flows does confine the model to use as a flood event simulator.

It was concluded from these tests that the enhanced RMA-2 scheme simulates those processes expected of it and that those processes interact in a logical way, consistent with the mathematical description. Furthermore, this model is able to adequately describe the specified flow problem in a stable and accurate fashion.

7.1.3 Operational rules for model applications

An application of the model to an 11 km reach of the River Culm, Devon, UK was undertaken to allow verification of model predictions against field data. As a consequence, a series of operational rules for future model applications was determined. These specify model stability limits in terms of topographic criteria but require independent testing for a further test reach to fully confirm their applicability.

7.1.4 Predictive capability of the enhanced RMA 2 scheme

The predictive capability of the model has been assessed in respect of two criteria; the ability of the model to reproduce observed results at the downstream system outlet and its ability to predict distributed flow field information over the entire reach. This latter capability is unique to the model developed in this thesis.

7.1.4.1 Predictive capability at the downstream system outlet

RMA-2 predictions of stage and discharge were compared to observed process behaviour at the downstream outlet of the River Culm study reach for a range of flood events, varying in both magnitude and complexity. Initial dynamic simulations were developed for this reach on the basis of a physically derived parameter set. Subsequently, a series of numerical experiments were run to determine the impact on model predictions of:

i) model calibration, based on the most significant parameters identified by the sensitivity analysis discussed above,

ii) altering the configuration of the downstream control structure,

iii) imposing stage flow boundary conditions at the downstream extremity of the mesh.

This has allowed the model's predictive capability at the downstream system outlet to be maximised for given levels of data availability. It was concluded from these tests that model generated results at the downstream system outlet showed a good level of correspondence to field data. **The scheme is therefore capable of accurately simulating a dynamic flood event.** These predictions were consistent with the theoretically derived accuracy estimates reported in Section 8.1.2.

7.1.4.2 Predictive capability over the entire reach

The entire predictive performance of the enhanced RMA-2 model has been assessed by comparing model predicted maximum inundation to field data. This was derived from ground and air photos and supported by post event surveys of trash lines on the floodplain. Data for two floods were obtained. These represented immediately out-of-bank (1 in 1 year event) and near totally inundated (1 in 5 year event) conditions. The simulations undertaken demonstrated that the enhanced RMA-2 model was able to explain between 60% and 90% of the variation in the observed maximum inundation data set.

This research has successfully generated a two dimensional finite element model with the capability to predict flow field information, such as water depths, velocities and inundation, over the entire study reach. Furthermore, the model is able to resolve flow structures having horizontal length scales of the order of 10 - 100 m. This unique development therefore represents a considerable extension of the capabilities of this class of hydraulic model. Moreover, significant potential is indicated in physical hydrology for model evaluation methodologies that explicitly consider 'internal' process representation.

7.2 Realisation of modelling objectives

The primary objective of this thesis, stated in Chapter 1, was the development of a modelling capability to simulate river channel/floodplain flow at a high level of spatial and temporal resolution for reach lengths appropriate to floodplain inundation. A number of design criteria for such a scheme were identified. It was concluded that the scheme must:

i) have parsimonious requirements for data,

ii) be capable of simulating water depths, velocities and inundation over horizontal scales of 10 - 100 m,

iii) be capable of dynamic simulations,

iv) be able to account for complex topography.

The discussion in Section 7.1 has demonstrated that as a result of the modelling developments reported in this thesis these objectives have been fulfilled in every respect.

Previously available schemes for simulating river channel/floodplain flow at the long reach scale, identified in Section 2.2.2.1, are based on the one dimensional St. Venant equations. However, flow representation in this class of model is ultimately unrealistic when compound channels are considered. In such schemes floodplains are either considered part of a single channel, treated as storage areas or as a discrete routing zone (see Figure 7.2a - c respectively). Recent studies have shown floodplain

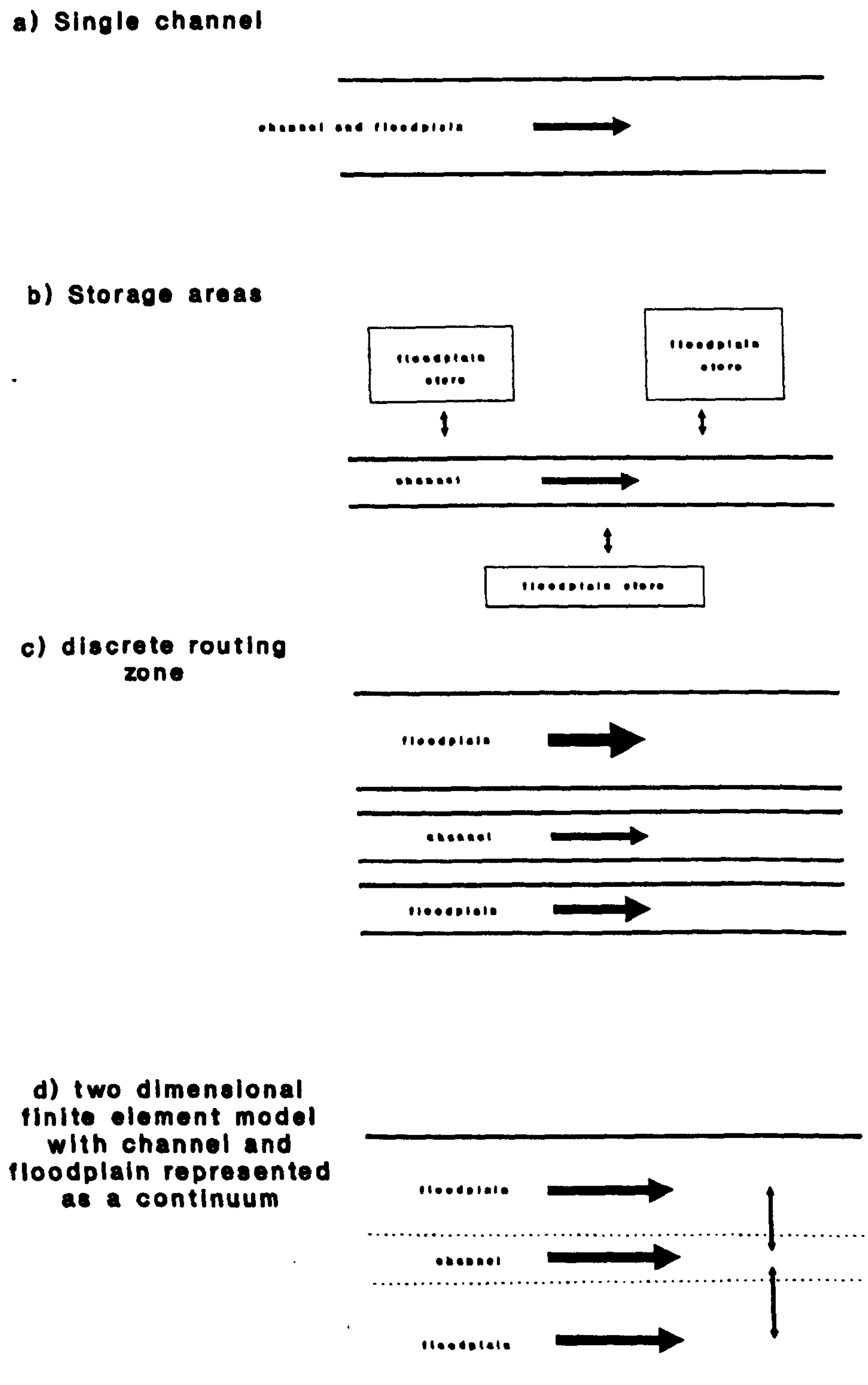


Figure 7.2: Representation of floodplain areas in one and two dimensional flow modelling schemes applicable to long reach scale problems. Note the significantly more realistic representation offered by 2-D methods.

flow to be an inherently complex media, which incorporates a number of dynamically interacting processes operating within a river channel/floodplain continuum. The model presented in this thesis represents the first scheme capable of simulating these processes at the long reach scale (see Figure 7.2d). The development of this unique capability, identified in Section 1.1 as essential to a wide range of disciplines, therefore represents the realisation of this thesis's primary objective.

This Chapter has outlined the satisfactory completion of the research objectives outlined in Chapter 1. Having summarised current model performance capabilities, it is now possible to proceed with extension of this scheme via the development of model application scenarios. In particular, Chapter 6 has identified that to fully exploit the model's potential to make distributed predictions of the flow field parameters, this aspect of model performance requires further confirmation against a third independent data set. The objective of Chapter 8 is therefore to undertake model applications that, firstly, provide such validation and, secondly, demonstrate the potential of such distributed schemes.

CHAPTER 8

Extending the two dimensional finite element scheme

In the preceding Chapters a model with the capability to simulate flow processes over long reach lengths in compound meandering channels has been developed and evaluated. This evaluation procedure has shown model performance to be satisfactory for known levels of parameter estimation error. Furthermore, it has been demonstrated that the internal process behaviour predicted by the model is a reasonable reproduction of known physical system states for a range of flow and topographic conditions. Evaluation of the enhanced RMA-2 modelling scheme has therefore advanced to the stage where it is now appropriate to consider extension of the model.

Extension of the modelling capability outlined above can be achieved in two ways; via enhancement of the numerical model, which will be briefly discussed in Section 8.1, or via the development of model application scenarios, discussed in detail in Section 8.2. Both methods are equally valid. However, given the wide potential for floodplain inundation modelling outlined in Section 1.1, this thesis will begin initial studies into a further phase of model development via this latter, application-based, route. In particular Chapter 6 has identified the need to more fully confirm the ability of the enhanced RMA-2 scheme to make accurate distributed predictions of flow field information over the entire reach. This extension of the modelling capability is therefore the primary objective of this Chapter.

8.1 Numerical model developments

Four specific changes to the physical model can be identified through which the modelling capability can be extended. These are:

i) To determine the precise relationship between mesh topographic resolution and micro-scale flow features. This task has relevance in a number of areas, particularly in the context of geomorphological investigations of floodplain environments. One approach to the investigation of this relationship is via the construction of experimental floodplain topographies of varying heterogeneity in a Digital Terrain Model (DTM) format. These should cover the observed variability range of floodplain microtopography. Finite element discretizations may then be imposed and a two dimensional matrix of model simulations constructed to allow the impact of discretization scale on flow prediction to be quantified.

ii) To enhance the wetting and drying algorithm within the current numerical scheme. The current formulation in the finite element model is based on a single physically based elemental volume coefficient, derived from the subgrid bathymetry. This is identical for both the wetting and drying phases of the inundation process. In reality, inertial effects and ponding, as well as changes in boundary friction with depth, will create differences in process behaviour between these two states. Significant numerical development of this algorithm is therefore necessary to include the processes identified above.

iii) To resolve constriction problems caused by flow control structures at inflow and outflow points on the finite element mesh. This is necessary to give accurate inundation predictions over the entire reach.

iv) To incorporate hillslope inflows into the model. Current hydraulic models assume that the contribution of such flows is negligible (Slade and Samuels, 1990). However, treating the floodplain as a 'black box' may be an inappropriate assumption when such models are applied over long reaches. Here the volume of such flows may be significant. Research is required to assess the relative importance of such contributions and, if necessary, to proceed with the development of a combined hillslope hydrology/floodplain flow model.

As a result of model developments (i) - (iv) above, the scheme's ability to accurately resolve floodplain velocity vectors and depths for a range of environments and flow conditions would be further enhanced.

One further modelling initiative, relating to the evaluation of complex environmental models in general, has also been identified by this project. High space and time resolution models, such as the enhanced flow modelling scheme developed in this thesis, produce extremely large results files, particularly for dynamic simulations. It is often impossible to evaluate this data set as a whole and the modeller is forced into a series of arbitrary decisions in order to reduce this complexity to a manageable level. In this context, video post processing of simulation graphics may present a means of displaying the **entire** set of results for a particular simulation. This would provide a more complete means of identifying numerical irregularities in model predictions and enable simulations with extremely small time steps (c. 60 seconds) to be evaluated.

8.2 Development of model application scenarios

Having briefly discussed a number of numerical model developments that could form the basis of future research, the remainder of this Chapter will concentrate on the development of model application scenarios. Chapters 6 and 7 have identified that the primary task for any further phase of development should be confirmation of the model's ability to predict distributed flow field information. This would allow the full potential of the model developed in this thesis to be exploited. Two applications were therefore selected; an initial study to provide further model validation and a second where distributed flow field information could be linked to other environmental models to provide an example of their potential. A number of applications with the ability to fulfil these requirements were identified. These included:

i) assessment of flood defence programmes,

ii) assessment of flood risk and floodplain land use planning,

iii) impact assessment of engineering structures on the floodplain,

iv) analysis of floodplain geomorphological processes,

v) development of floodplain sediment deposition theories,

vi) definition of ecological zones on the floodplain using such depth/velocity indices as the Instream Incremental Method (Bullock and Gustard, 1992),

vii) validation of simple routing models using the RMA-2 scheme as 'ground truth' (see for example Baird *et al.*, 1992).

From these, flood risk assessment and the investigation of sediment deposition processes within floodplain systems, were selected as appropriate to the research objective identified above. Model application to flood risk assessment necessarily entails the construction of additional sections of finite element mesh. This will allow confirmation of the operational procedures for network construction identified in Chapter 5 and, more importantly, will further extend the process of validating internal process predictions. Having established the validity of these against a third, independent, data set, confidence in distributed predictions derived from RMA-2 will be sufficient to make linkage to other models possible. Thus, the preparatory work undertaken in Section 8.2.1 is an essential precursor to the initial exploration of RMA-2's ability to investigate floodplain depositional process, as this requires accurate information on distributed flow field parameters. The future research directions investigated in this Section therefore encompass key issues whose resolution is relevant to a number of other applications and also serves to represent concerns highlighted in previous Chapters. In summary therefore, the objective of this research is to:

i) allow refinement of the existing modelling capability,

ii) provide examples of the utility of the developed model,

iii) serve to reinforce the conclusions drawn in this thesis.

8.2.1 Flood risk assessment

As stated in Section 1.1.1 floodplain environments are under increasing pressure from developers. In certain highly populated areas, such as South Eastern England, floodplains may represent the only available building land. Given this development pressure on planners, environmental managers and engineers, the need for accurate models of floodplain flow is increasing, as is the range of prediction products demanded. Current analytic tools for this problem are typically based on a solution of the one dimensional St. Venant equations (Samuels, 1990) in conjunction with an appropriate numerical approximation procedure (see Section 2.2.2.1). Data needs for such models are intensive and recourse must often be made to data collection programmes involving detailed topographic survey, flow monitoring and site inspection. These substantially increase the time and resources needed to complete any modelling study, although the resulting model is likely to be extremely accurate. Despite this accuracy, one dimensional schemes make no account of lateral variations in the flow field and therefore may be incapable of providing certain information types required by engineers. These may include identification of dynamic inundation zones and flow lines throughout flood events. Moreover, changes to the topographic discretization employed by such models will often require further topographic survey, model reconfiguration and calibration. The ability to easily assess the impact of proposed developments is thereby lost.

In order to meet flood risk assessment demands currently being made on practising engineers it would clearly be of benefit if floodplain flow modelling were to be capable not only of generating accurate hydrographs for particular cross sections, but were also able to:

i) identify inundation zones, flow lines and water surface elevations over the entire reach,

ii) account for topographic complexity and be parameterized on the basis of extremely sparse data sets,

iii) allow rapid alteration of the topographic discretization to investigate the impact of proposed developments,

iv) produce accurate simulations on the basis of simple calibration procedures.

It will be argued here that these objectives are best achieved using a two dimensional finite element scheme of the type developed in this thesis. Objectives (i), (ii) and (iv) are made possible by the more complete description of the flow field provided by two dimensional flow equations. This allows a wider range of prediction products to be generated and reduces the reliance on artificial manipulation of model parameters to obtain a realistic simulation of a given flow field. Objective (iii) is achieved as a consequence of the finite element method's ability to utilize solution grids with variable resolution. It is therefore relatively easy to delete, deform or add elements in particular locations to represent proposed structures. This avoids the creation of an entirely new mesh.

The purpose of this Section is therefore fourfold:

i) to present an application of the developed two dimensional finite element scheme to a flood risk assessment problem,

ii) to provide examples of model output capabilities in respect of this application,

iii) to determine the potential of the developed model to serve as a practical flow modelling tool for engineering problems.

iv) to use this application to further extend the RMA-2 modelling capability, in particular to further confirm the accuracy of the internal process representation.

8.2.1.1 Research design and experimental method

An appropriate flood risk assessment application for RMA-2 should ideally possess a number of characteristics. The application should:

i) necessitate further mesh construction,

ii) allow further validation of internal process predictions,

iii) preferably involve some engineering problem for which solutions are required.

An exploratory investigation identified a site approximately 3 km below the downstream extremity of the River Culm study reach described in Chapter 5 which fulfilled these criteria. This site was therefore selected as a first example of a long reach scale flood risk assessment problem investigated using two dimensional finite element techniques. Here a significant flood hazard exists for a village, Stoke Cannon, built directly on the floodplain near to the confluence of the River's Culm and Exe. In addition to locational factors, the flood hazard at Stoke Cannon is exacerbated by a constriction of the floodplain at this point and by possible backing up effects from the A396 road bridge which crosses the floodplain downstream of the village. In particular danger are a Primary School situated approximately 30 m from the channel and an adjacent residential area. As a consequence of regular flooding, the National Rivers Authority (NRA) have constructed a flood alleviation scheme for this site consisting of a combination of culverts and retaining walls. In order to design and manage such a scheme, practising engineers have a number of information requirements which may typically include:

(i) design information concerning the standard of protection offered by the alleviation scheme,

(ii) operational flood warning information concerning upstream stage criteria that warn for overtopping of this structure,

(ii) standards of service information concerning the magnitude and timing of floodplain occupation by discharges of various frequencies.

Derivation of these information types from two dimensional finite element model data was undertaken in an initial study consisting of five phases:

(i) extension of the existing River Culm finite element mesh,

(ii) data provision for model simulations,

(iii) calibration and validation,

(iv) demonstration of predictive products relevant to flood risk assessment as outlined above,

(v) determination of predictive uncertainty.

Extension of the existing River Culm finite element mesh was undertaken using the operational rules derived in Chapter 5. This application therefore affords an opportunity to independently test these rules on a separate floodplain reach. Although the reach selected is essentially similar in many respects to the preceding upstream section, significant differences do exist in terms of slope, floodplain width and in the variety of complex topographic features present. These differences were felt to be sufficient to provide a rigourous test of the developed procedure. Topographic discretization for this mesh was made on the basis of UK Ordnance Survey 1:25 000 and 1:2500 series maps in addition to a surveyed cross section taken at the downstream extremity of the extended reach which precisely fixed bed and floodplain elevations at this point. The resulting finite element mesh extension (see Figure 8.1) consists of 940 nodes and 450 elements and incorporates such topographic features as the flood alleviation scheme at Stoke Cannon, the Stoke Cannon Primary School and two road bridges which potentially act as impediments to flow. In particular, specific nodes representing the former two features, with bed elevations determined from NRA site drawings, were included within the mesh to allow prediction products to be extracted at these points (see Figure 8.2). The resulting combined mesh therefore comprises approximately 4500 nodes and 1815 elements and represents 14 km of river reach.

Regarding data provision for model simulations, no gauging station exists at Stoke Cannon with which to construct a boundary condition for the numerical model. The modelling package HYMO3 (Baird and Anderson, 1992) was therefore used in conjunction with the surveyed cross section at Stoke Cannon to approximate a stage discharge relationship for this site. This procedure is likely to be subject to some degree of inherent error, estimated to be up to a maximum of ±15%. All other model parameters, such as boundary friction, eddy viscosity and domain coefficients, were determined on the basis of site inspection, from physically based values determined for the application described in Chapter 5 or assumed from previously published studies.

As no flow data exists for Stoke Cannon, calibration and validation of the model at this point could not be undertaken. Consequently, these tasks were undertaken using available flow data for the Rewe gauging station. **This has the added advantage of providing further confirmation that the internal process behaviour predicted by the model is a satisfactory reproduction of field observations.** Calibration of such models has typically involved extensive manipulation of parameters, such as boundary friction, to ensure satisfactory levels of correspondence between observed and predicted results (see for example Evans and Lany, 1983). For this application, however, it has been possible to configure the model at a lower level of calibration.

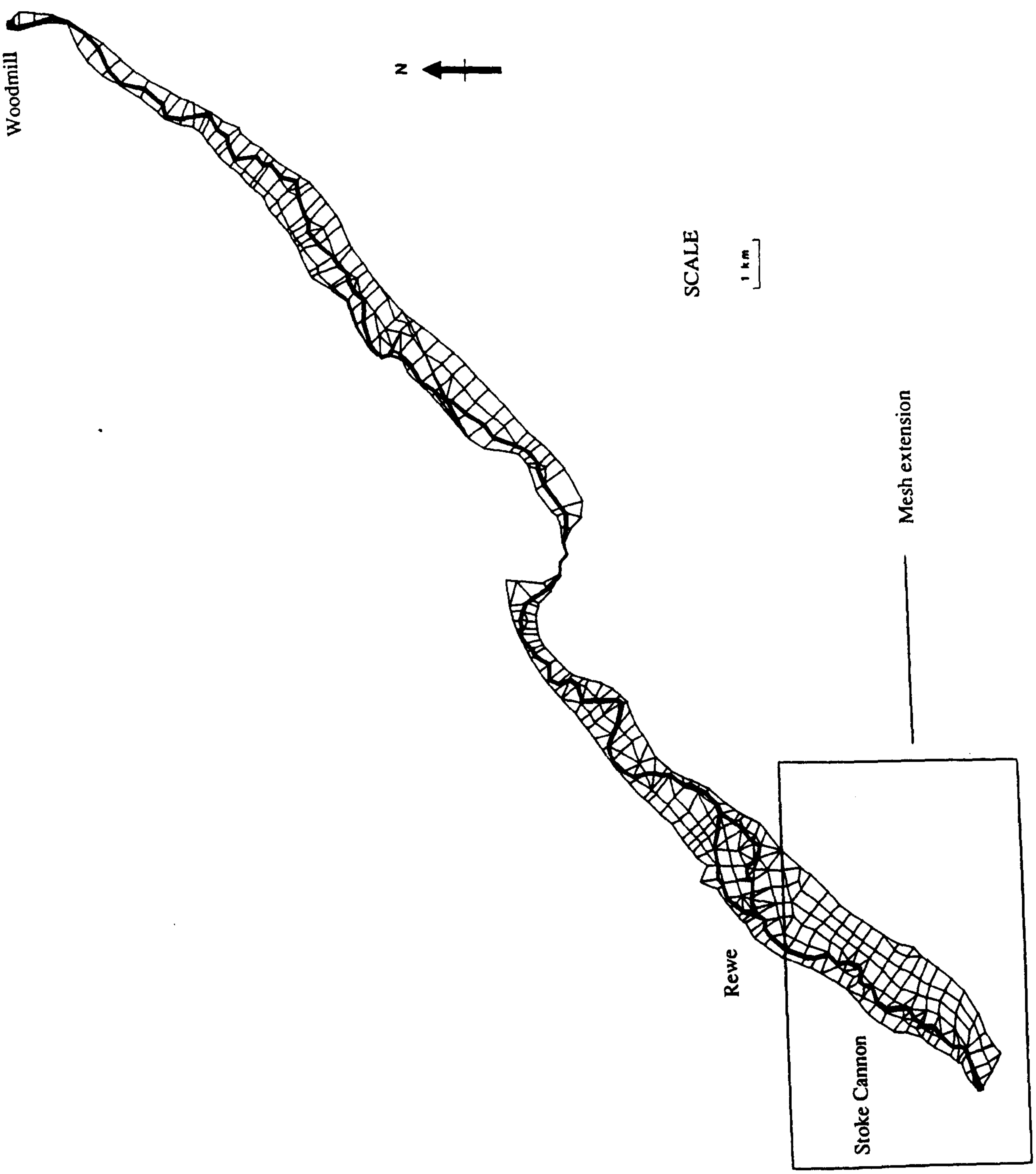


Figure 8.1: The 3 km extension to the River Culm finite element mesh configured to examine flood hazard in the vicinity of Stoke Cannon.

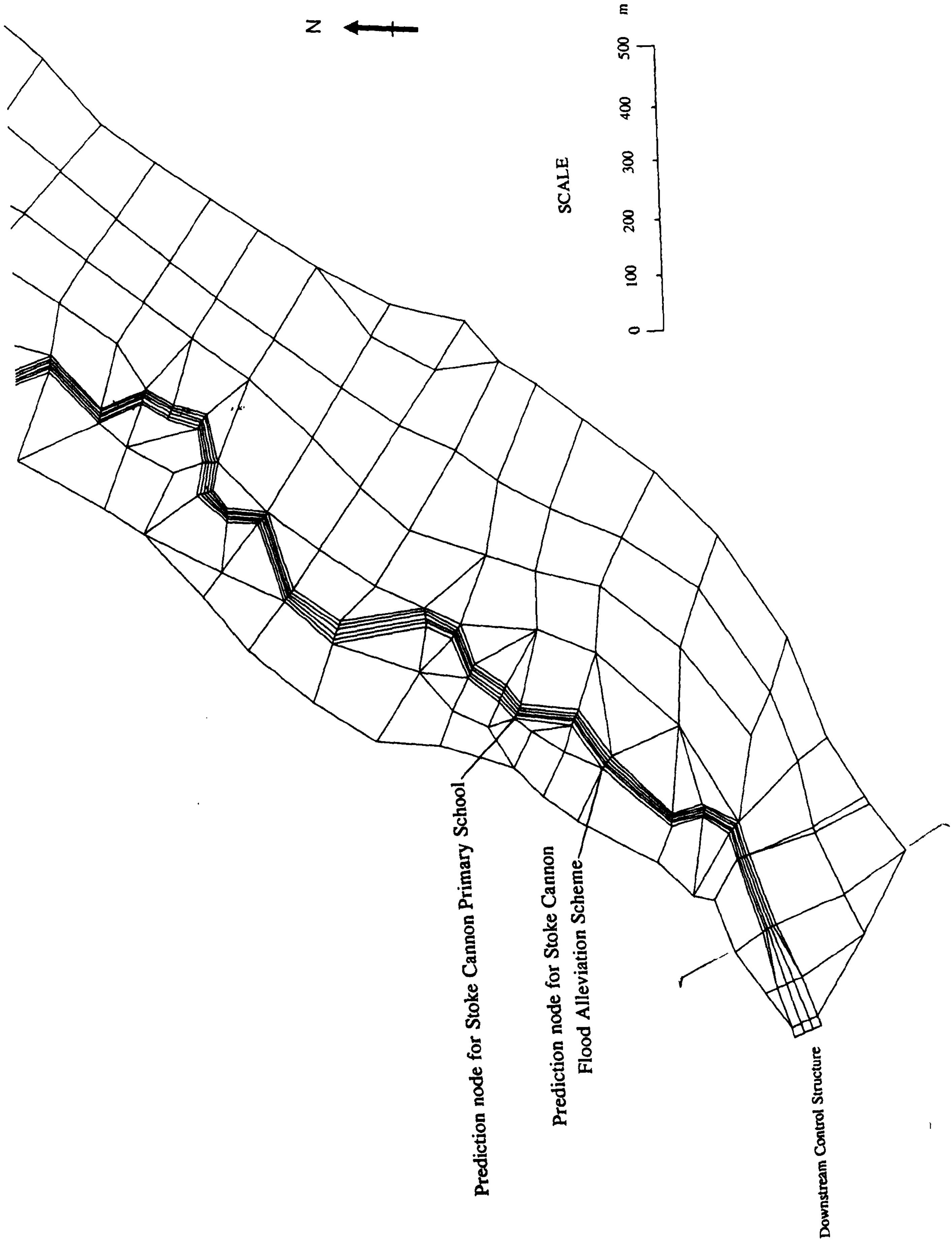


Figure 8.2: Detail of finite element mesh extension configured to represent Stoke Cannon and its associated flood alleviation scheme.

Specifically, only one further simulation in addition to that made using the initial parameter set was required to obtain sufficient prediction refinement. This is consistent with practical utilization of the model as set up times are reduced, the model is less restricted to particular flow or seasonal conditions, and alterations to the topographic discretization can be made without recourse to extensive periods of further calibration.

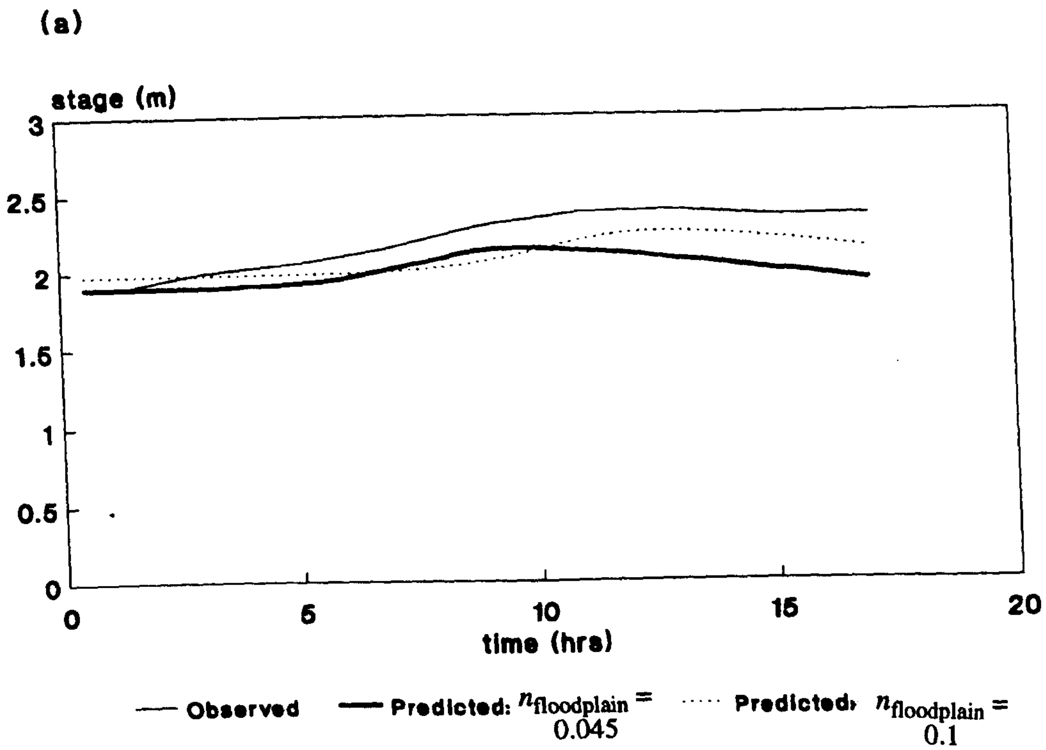
The model, initialized on the above basis, was used to simulate a series of flows having recurrence intervals of 1, 12, 100 and 1000 years. The 100 and 1000 year events were not available from the flow record at Woodmill and were therefore derived by scaling up the 1 in 12 year event using the established flood frequency relationship for this site. The data from these simulations were then used to derive a variety of prediction products.

Chapter 4 highlighted the variation in model accuracy due to parameter estimation error. The translation of this error into predictive uncertainty is of particular importance for the evaluation of this application due to the lack of flow data at Stoke Cannon for model validation. In order to assess this uncertainty, the final phase of this initial study consisted of undertaking further simulations of the 12, 100 and 1000 year events imposing an error of ±15% on the downstream boundary condition.

8.2.1.2 Simulation results

Validation of model simulations for the 1 and 12 year events, for which flow data exist at Rewe, is presented in Figure 8.3. Two simulations of each event have been carried out in accordance with the simple calibration procedure defined in Chapter 5. An initial simulation with a floodplain roughness coefficient of n = 0.045 and 0.055 was refined by increasing this parameter value to 0.1. This was sufficient to give a satisfactory reproduction of the observed field data. Although this latter value of n appears to be at the upper end of the range of field derived estimates noted both for this and other floodplains, it does give a better approximation to the observed data. Possible reasons for this are examined in Section 8.1.3.

Predicted hydrographs at Stoke Cannon, with an error band of ±15%, are shown for 12, 100 and 1000 year recurrence interval events in Figure 8.4. For these simulations no overlapping of predicted error bands is evident. This demonstrates that even if



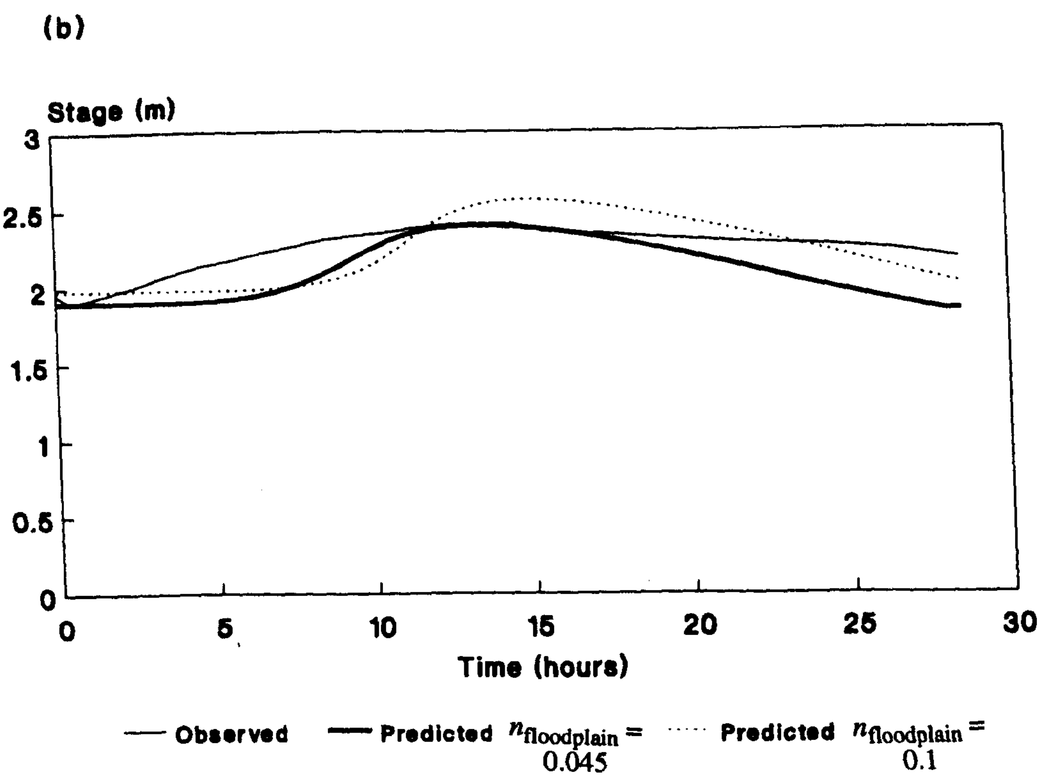


Figure 8.3: A comparison of observed stage hydrographs for the Rewe gauging station and RMA-2 simulations using $n_{floodplain}$ = 0.045 and 0.1. Two events are shown; a 1 in 1 year (Figure 8.3a) and a 1 in 12 year event (Figure 8.3b). This independently confirms the enhanced model's ability to make spatially distributed predictions of flow field parameters.

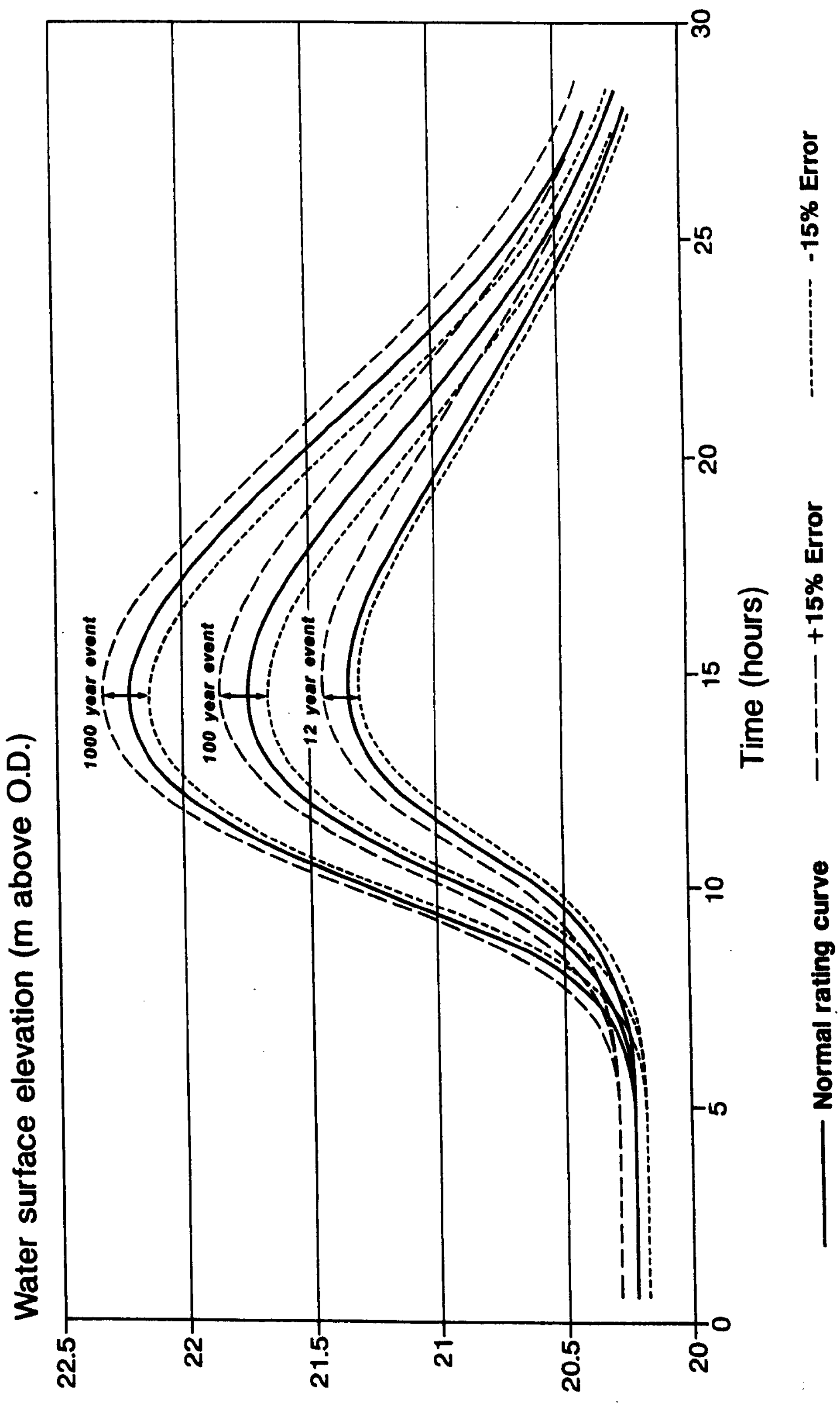


Figure 8.4: RMA-2 predicted stage hydrographs at Stoke Cannon for 12, 100 and 1000 year events, showing prediction variation due to rating curve estimation error.

significant errors exist in the downstream boundary condition estimation procedure the impact on model results is relatively low.

Having confirmed that model simulation results are an adequate reproduction of the physical system, it is possible to use predicted depth and velocity at each node to provide hydraulic information concerning flood hazard that will be of use to practising engineers. Three such information classes relating to design, operational flood warning and standards of service have previously been outlined in Section 8.1.1. In order to demonstrate the scope of model generated prediction products, worked examples for each of these problem classes will now be given. Although both design and flood warning data could be derived with existing modelling schemes a two dimensional finite element model has a number of significant advantages in this context. Most importantly, the scheme is able to accurately represent complex topography around the Stoke Cannon flood alleviation scheme that one dimensional models could not readily take into account. Such a topographic discretization is essential for a satisfactory description of the flow problem. Moreover, information concerning the timing of floodplain occupation by discharges of various frequencies can only be provided by a two dimensional finite element scheme.

In terms of the construction of flood alleviation structures one particularly important design criteria is the discharge frequency at which overtopping occurs. Accurate determination of the standard of protection afforded by a particular structure will allow problems of over- or under-design to be eliminated. For Stoke Cannon, the overtopping elevation of the flood alleviation scheme retaining wall was determined from NRA site drawings. Maximum water surface elevation data was then determined for each discharge frequency at the specific node in the finite element discretization configured to represent the scheme. This data is presented in Table 8.1. The model therefore predicts that the standard of protection offered by this retaining wall is < 1 in 12 years.

Height of wall	Maximum predicted stage
21.03 m	20.82 m: 1 in 1 year event 21.29 m: 1 in 12 year event

Table 8.1: Water surface elevation for the Stoke Cannon Flood Alleviation scheme

Operational flood warnings for this site are typically made with respect to flooding of the Stoke Cannon Primary School. Particular stage thresholds are set for the upstream gauging sites at Woodmill and Rewe whose exceedence indicates incipient flooding. Currently these are 3.2 m and rising at Woodmill and 2.6 m and rising at Rewe. These limits are selected by NRA Flood Forecasting Officers on the basis of their personal experience. Given such an *ad hoc* system, considerable scope exists for using appropriate physical modelling techniques to determine these thresholds. This is however dependent on configuring models, such as the one developed here, that can accurately predict water surface elevations along the whole reach. For this specific application the predicted discharge frequency of flooding at the Primary School was determined in the manner outlined above (see Table 8.2). This indicated that flooding would occur for the 1 in 12 year event. Maximum stage at Woodmill and Rewe was then determined for this event from observed flow records and model predictions. This data is given in Table 8.3. These predictions are consistent with existing flood warning criteria and may indicate some scope for reduction of these warning thresholds. However, a more accurate determination of the elevation at which the Primary School floods and an examination of the impact of hydrograph shape on the flood hazard would be necessary precursors to such a decision.

Height of wall	Maximum predicted stage
21.33 m	Not inundated: 1 in 1 year event 21.53 m: 1 in 12 year event

Table 8.2: Water surface elevations for the Stoke Cannon Primary School.

Gauging station	Stage
Woodmill	3.05 m
Rewe	2.39 m

Table 8.3: Upstream stage criteria to warn of flooding at Stoke Cannon Primary School by a 1 in 12 year event.

The standards of service approach to management of river channel/floodplain reaches is a relatively recent development (see for example Jaeggi and Zarn, 1990). The approach, for a particular section of floodplain, relies on determining the inundation extent for particular flows and relating this flood risk to some notional assessment of the 'value' of that area. In this context the ability to predict the extent and temporal dynamics of inundation, such as might be provided by the RMA-2 model, is of critical importance. For this study the section of floodplain between Rewe and Stoke Cannon has been selected and the percentage floodplain occupation for each event determined. This information is shown in Table 8.4, whilst the maximum inundated area for the 1 in 1 year event is shown in Figure 8.5. We may therefore state that the standard of service provided along this reach is < 1 in 12 years with a standard of < 1 in 1 year for certain near channel areas. Despite the limitations of this first pass study, the potential of two dimensional finite element models to contribute to standards of service assessment programmes would appear to be high.

Event recurrence interval	% Floodplain occupation
1 in 1 year	67.5 (see Figure 3)
1 in 12 year	100
1 in 100 year	100
1 in 1000 year	100

Table 8.4: Predicted floodplain occupation with event frequency.

8.2.1.3 Discussion

This Section has attempted to demonstrate the potential of two dimensional finite element models to serve as a practical flow modelling tool for engineering problems. The example outlined above has confirmed that such a potential exists and that the resources required, in terms of set up and computing time are reasonable. The number of floodplain reaches where it is cost-effective to implement modelling

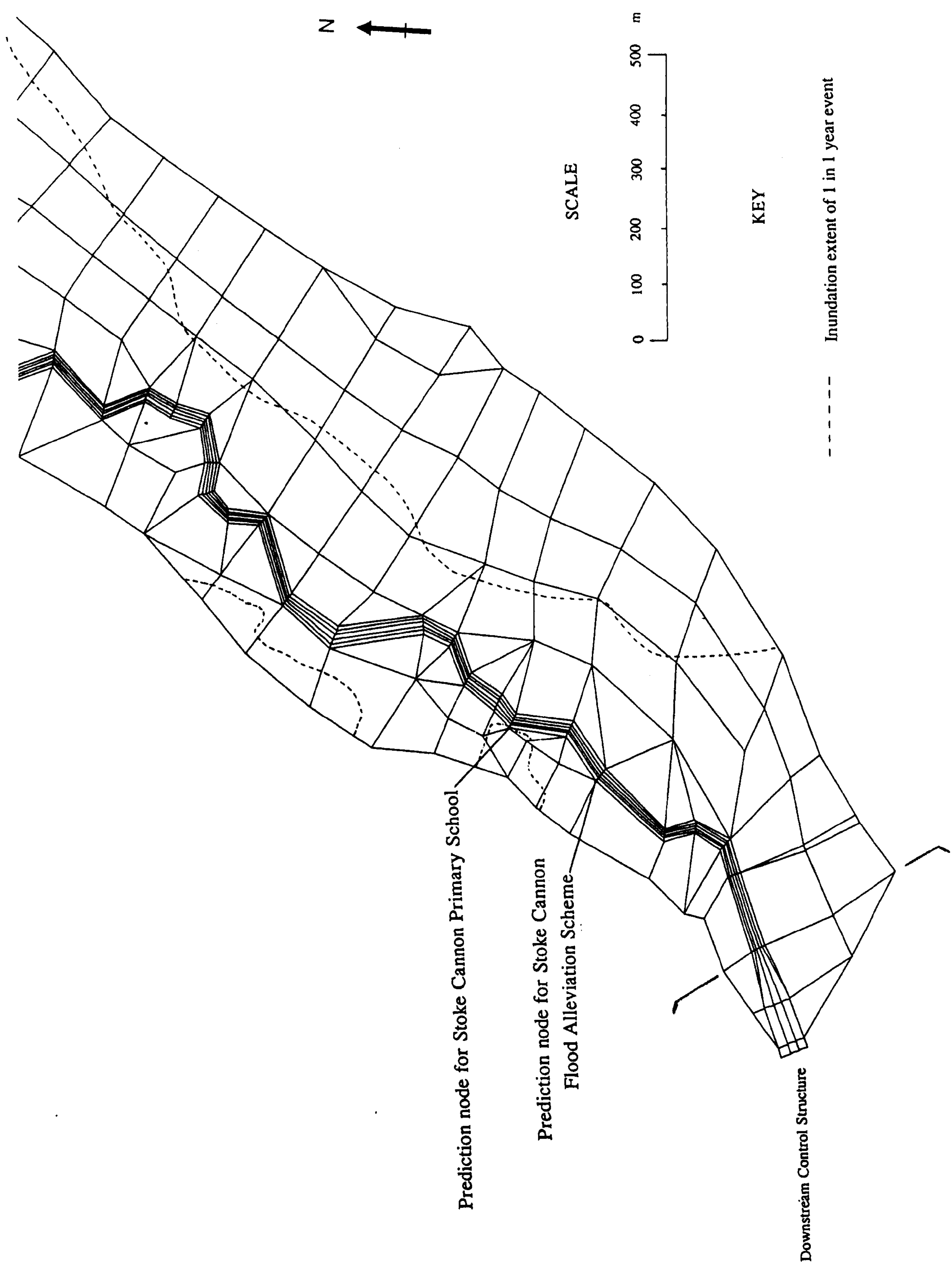


Figure 8.5: Detail of the finite element mesh in the vicinity of Stoke Cannon showing maximum predicted inundation extent of a 1 in 1 year recurrence interval flood.

studies is thereby increased. For example, model solutions can be constructed for rural reaches where a low level flood hazard would not have previously justified a fully surveyed one dimensional model. This does not conflict with the application niche of one dimensional approaches, as it is anticipated that such methods will still be necessary for problems where greater precision is required. There is scope, therefore, for the inclusion of two dimensional finite element solutions in the spectrum of approaches available for the analysis of floodplain inundation. As a consequence of this, a question arises of matching modelling approach to engineering needs.

This decision has previously been made qualitatively, however an increase in the number of modelling approaches available will require a more structured decision support system procedure to be developed. Progress towards such a system will be achieved by the pursuit of four research objectives for each model type under consideration. Firstly, each modelling type will require validation against a wide range of conditions and calibrations. Secondly, the range of prediction products that each model type can generate should be explored. Thirdly, on the basis of this information the range of problems to which each model type applies can be defined. Lastly, and most critically, the relationship between data provision and model accuracy must be quantified. An initial attempt to undertake such research is presented in this (Figure 8.4), however much further work is required, especially in relation to topographic data. This research should have the objective of constructing a decision support system able to define the most cost effective solution for a particular modelling problem, i.e. a model that is neither under- nor overspecified.

A further issue relating to the relatively high floodplain roughness coefficient values used to achieve calibration of model simulations also warrants further consideration. Beven (1989) has pointed out that no theoretical framework exists for the lumping of sub-grid processes for spatially heterogeneous grid squares. The physics upon which the controlling equations of distributed hydrological and hydraulic models are based are derived from the small scale physics of homogeneous systems. In constructing models beyond such spatial and temporal scales this homogeneity no longer applies. A degree of 'lumping' is therefore inherent in any distributed hydrological model. Single parameters are consequently used to effectively represent a large scale heterogeneous parameter distribution. An identical argument can be applied here in the case of boundary friction values. The single value of n used to represent grid scale boundary friction effects is a spatially averaged figure incorporating a number of time dependent effects, such as the change in boundary friction with depth over the course of a flood event. The effective value for boundary friction at the grid scale is therefore drawn from a different population of values than one would expect for an at-a-point measurement. The higher value of floodplain boundary friction required to calibrate the model is potentially a consequence of such a mechanism, although such arguments have not been considered in relation to hydraulic models. This explanation is consistent with the results of a recent series of field experiments. Beven and Carling (1992) showed that values of boundary friction estimated for reach lengths of 1-2 km using dye tracer techniques were higher than equivalent at-a-point values. This was attributed to the presence of large scale flow structures within the reach which acted to retard flow. Specifically, it was hypothesized that momentum exchange across internal shear zones within the flow occurs between volumes of slow and fast moving water. Other potential explanations include an increase in floodplain boundary friction generated by hedges running laterally across the floodplain or that we are calibrating for a less than adequate representation of channel areas. At present we cannot distinguish between these possibilities and therefore much further work is required.

8.2.1.4 Conclusions

In respect of the utility of two dimensional finite element models for problems involving the assessment of flood risk the following conclusions can be drawn:

i) such models can provide reasonable predictions **over the entire reach** on the basis of extremely sparse data sets, particularly in terms of topographic data,

ii) the operational rules for model applications identified in Chapter 5 are robust,

iii) a wide range of prediction products can be generated,

iv) the inclusion of two dimensional finite element models as a standard simulation technique for engineering problems is indicated,

v) a need has been identified to provide a decision support system to help practising engineers define the most cost effective solution for particular modelling problems.

8.2.2 Sediment deposition processes within floodplain systems

Having independently verified the conclusions drawn in Chapter 6 concerning the validity of distributed process predictions generated by the RMA-2 model, it is now possible to proceed with linkage to other models. Specifically, the ability of RMA-2 to investigate sediment deposition processes within floodplain systems will be considered.

Geomorphological investigations of floodplains have traditionally focussed on their long-term morphological and sedimentological evolution (e.g. Leopold and Wolman, 1957; Blake and Ollier, 1971; Lewin, 1978). In recent years, however, there has been a growing interest in the dynamics of contemporary floodplain systems, both in relation to the interaction between floodplain morphology and floodplain flows (e.g. Knight, 1989) and to detailed patterns of floodplain sedimentation (e.g. James, 1985; Walling and Bradley, 1989). Interest in the behaviour and fate of sediment transported by over-bank flows has necessitated a shift in both the temporal and spatial scales of investigation, relative to more traditional perspectives, and has introduced major new requirements for background data. Whereas generalised information on river stage, patterns of inundation, flood recurrence intervals and flow velocities may have sufficed for the interpretation of the long-term evolution of a floodplain, more detailed information on flow depths, inundation sequences and velocity vectors are frequently required to permit detailed analysis of short-term floodplain sedimentary dynamics. In particular recent advances in the documentation of rates and patterns of floodplain sedimentation (e.g. Walling *et al.*, 1992) have demonstrated a capacity to map local patterns of deposition which needs to be matched by detailed information on velocity vectors for effective interpretation.

Acquisition of the necessary detailed information concerning flow depths, velocity vectors and associated parameters is, however, subject to serious practical difficulties. The transient and unpredictable occurrence of flood events often means that the technical capability to undertake frequent measurements over an extensive network of measuring sites may not exist. Furthermore, magnitude and frequency considerations demand that any measurements should be undertaken for a wide range of events. In the light of this there is a need to utilize models capable of simulating the behaviour of floodplain flows with a high degree of spatial and temporal resolution, in order to reduce reliance on direct field measurement.

Models capable of meeting these demands therefore need to be spatially distributed and capable of dynamic simulation of entire flood events. The identification of inundation zones and velocity vector predictions generated by the RMA-2 model have obvious potential to fulfil these objectives and provide a significant input to geomorphological studies of floodplain environments. Again the ability of two dimensional finite element models to include complex topographic features within the numerical discretization is of particular importance in this context. Micro-topographic variations have been identified as being of critical importance in determining spatial heterogeneities in observations of floodplain sediment deposition (Walling *et al.*, 1986). It is thus essential that any model formulation attempting an understanding of such processes should include this topographic detail. Two dimensional finite element models therefore represent the only available modelling solution that can be utilized in this context.

In order to demonstrate the potential of two dimensional finite element schemes, examples of model output are presented for the bifurcation section of the finite element mesh described in Chapter 5 (see Figure 8.6). These results refer to peak flow conditions of the 1 in 1 year recurrence interval event shown in Figure 5.6a and show, respectively, velocity vectors for each computational node (Figure 8.7) and these velocity vectors interpolated over a rectangular grid (Figure 8.8). The spatial variation in water depth produced by this model simulation is shown in Figure 8.9. Figure 8.9 also shows this variation to be significantly different to the pattern of bed elevation. This therefore highlights the need for two dimensional finite element schemes which, because of the inclusion of complex topography in the numerical discretization, are able to identify such variations in the floodplain flow field. A generalized description of the flow field information relevant to geomorphologists is given in Figure 8.10.

In the context of geomorphological studies, it is worthwhile to draw attention to the following aspects of the results presented in Figures 8.7 - 8.10. Firstly, further confirmation is obtained of the model's ability to predict differences in the flow field over quite small horizontal ranges (10 - 100 m). For example, we can delineate an area of shallow, low velocity flow created in the lee of the channel bifurcation (area **a**, Figure 8.9). This contrasts strongly with the adjacent portion of the floodplain (area **b**, Figure 8.9), where a body of high velocity water crosses an arm of the channel bifurcation and continues onto the floodplain island creating a region of relatively deep, high velocity flow. These predicted differences have strong implications for the

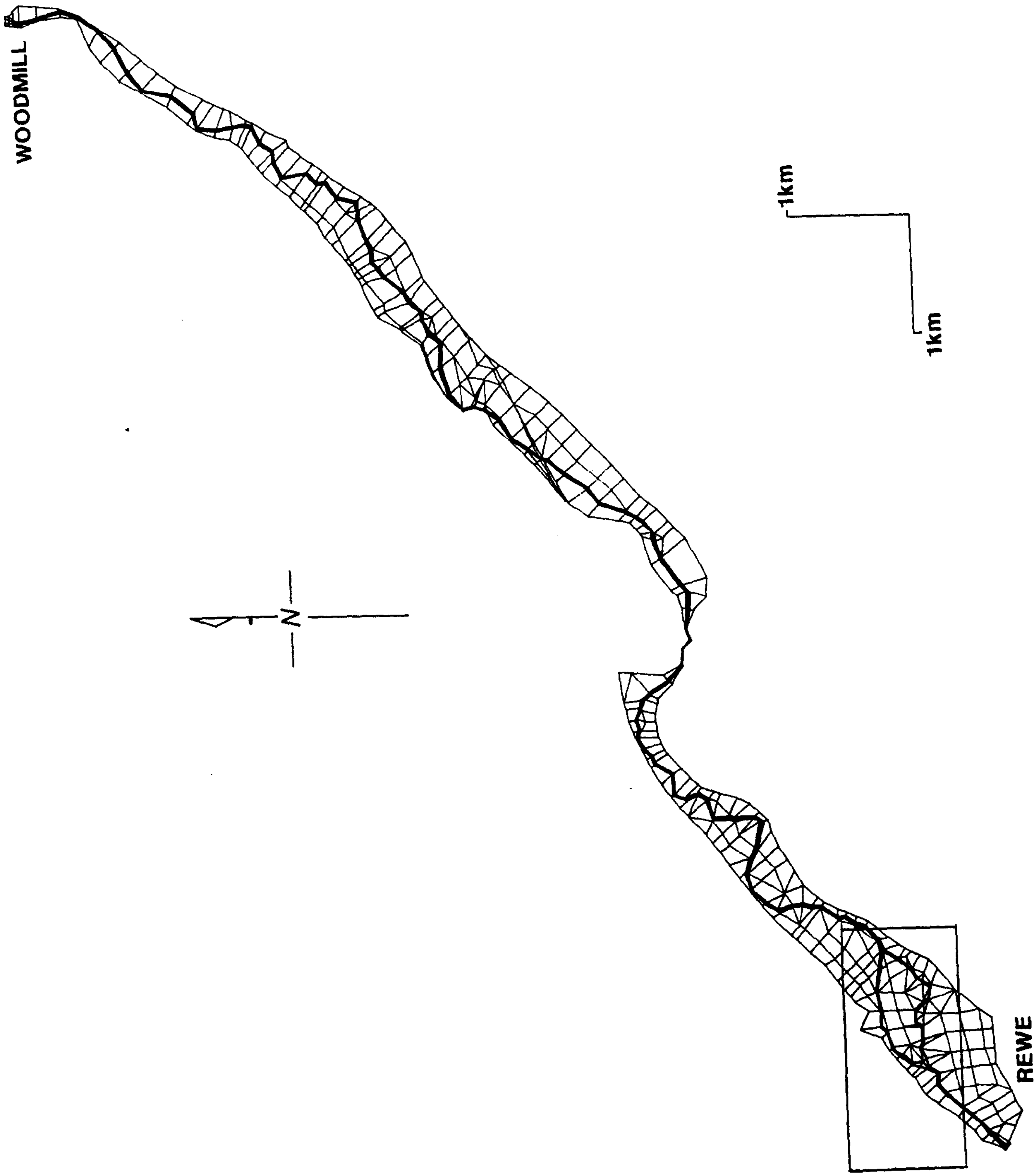


Figure 8.6: Finite element mesh configured for the River Culm study reach. The inset area delimits the detailed section of mesh used to demonstrate the model's applicability to sediment deposition problems.

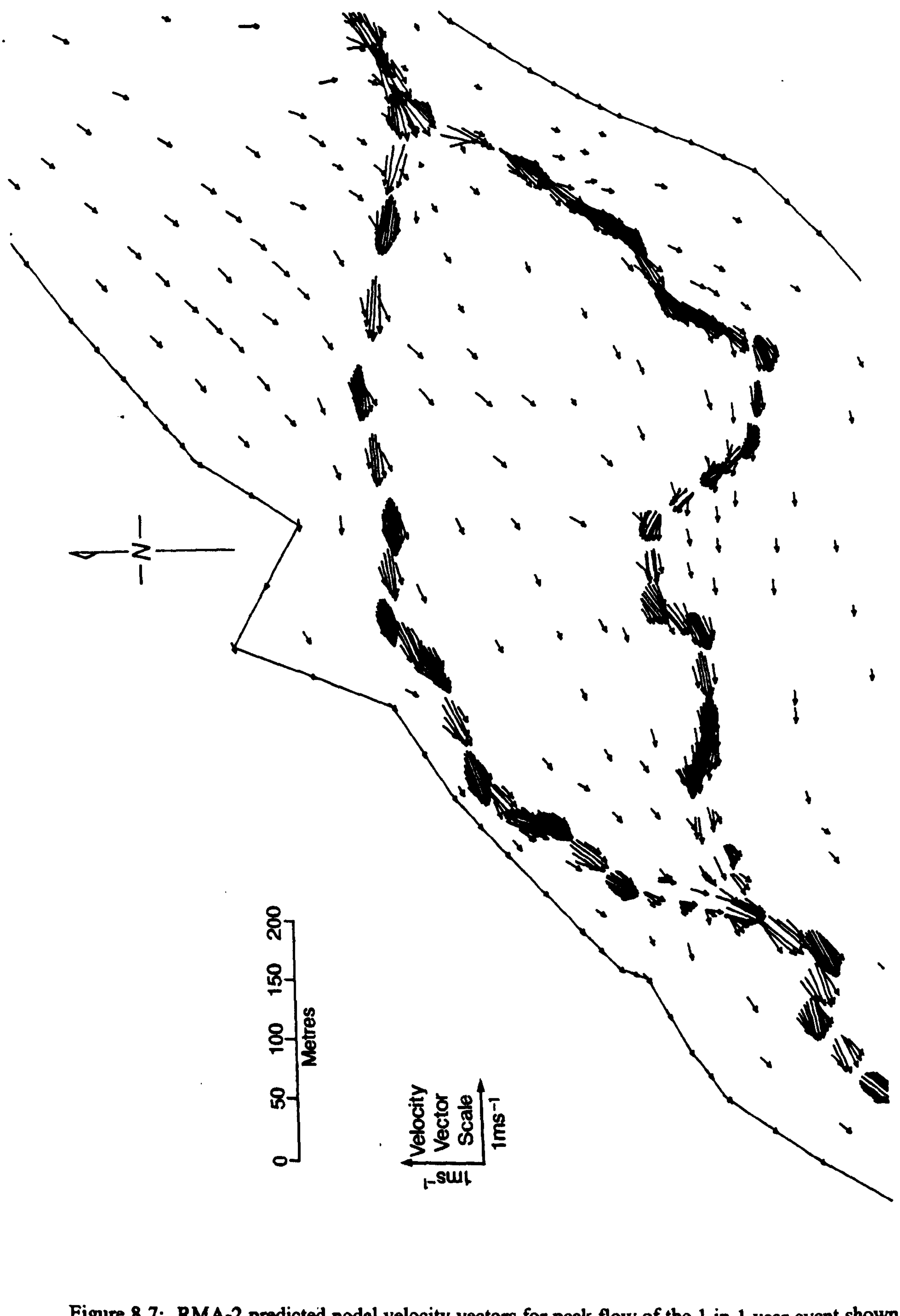


Figure 8.7: RMA-2 predicted nodal velocity vectors for peak flow of the 1 in 1 year event shown in Figure 5.6a.

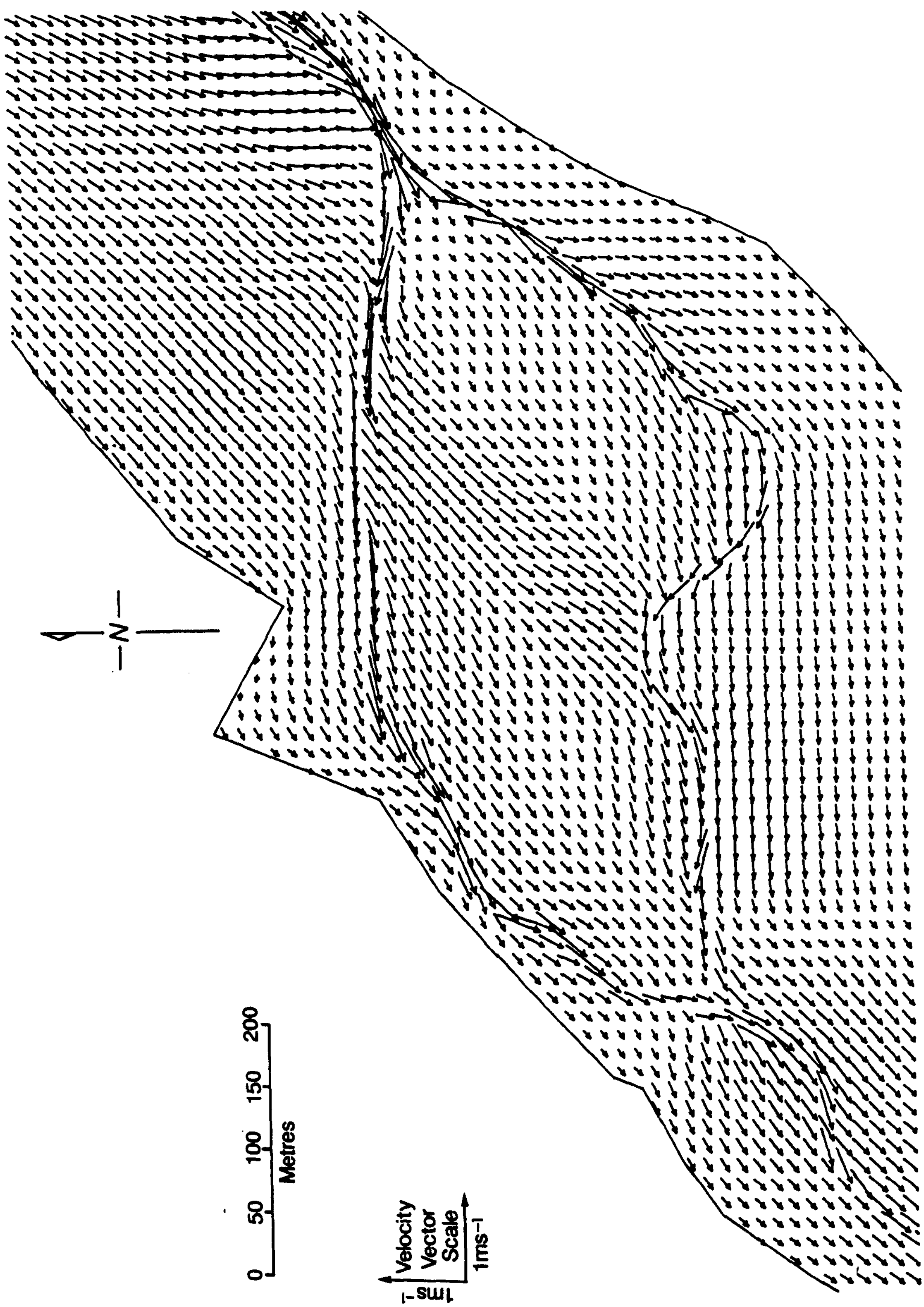


Figure 8.8: RMA-2 predicted velocity vectors interpolated over a rectangular grid for peak flow of the 1 in 1 year event shown in Figure 5.6a.

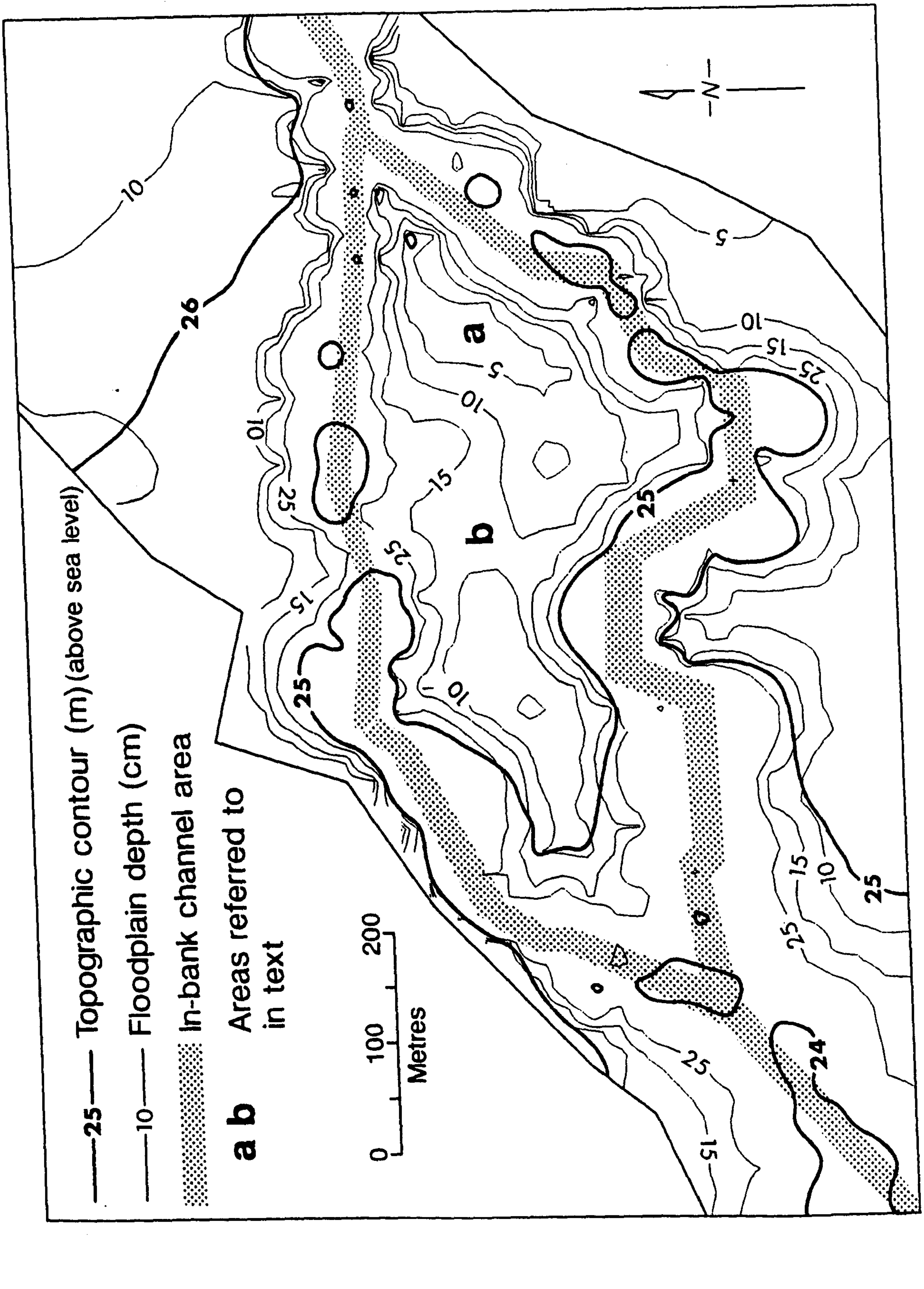


Figure 8.9: RMA-2 predicted water depths for peak flow of the 1 in 1 year event shown in Figure 5.6a.

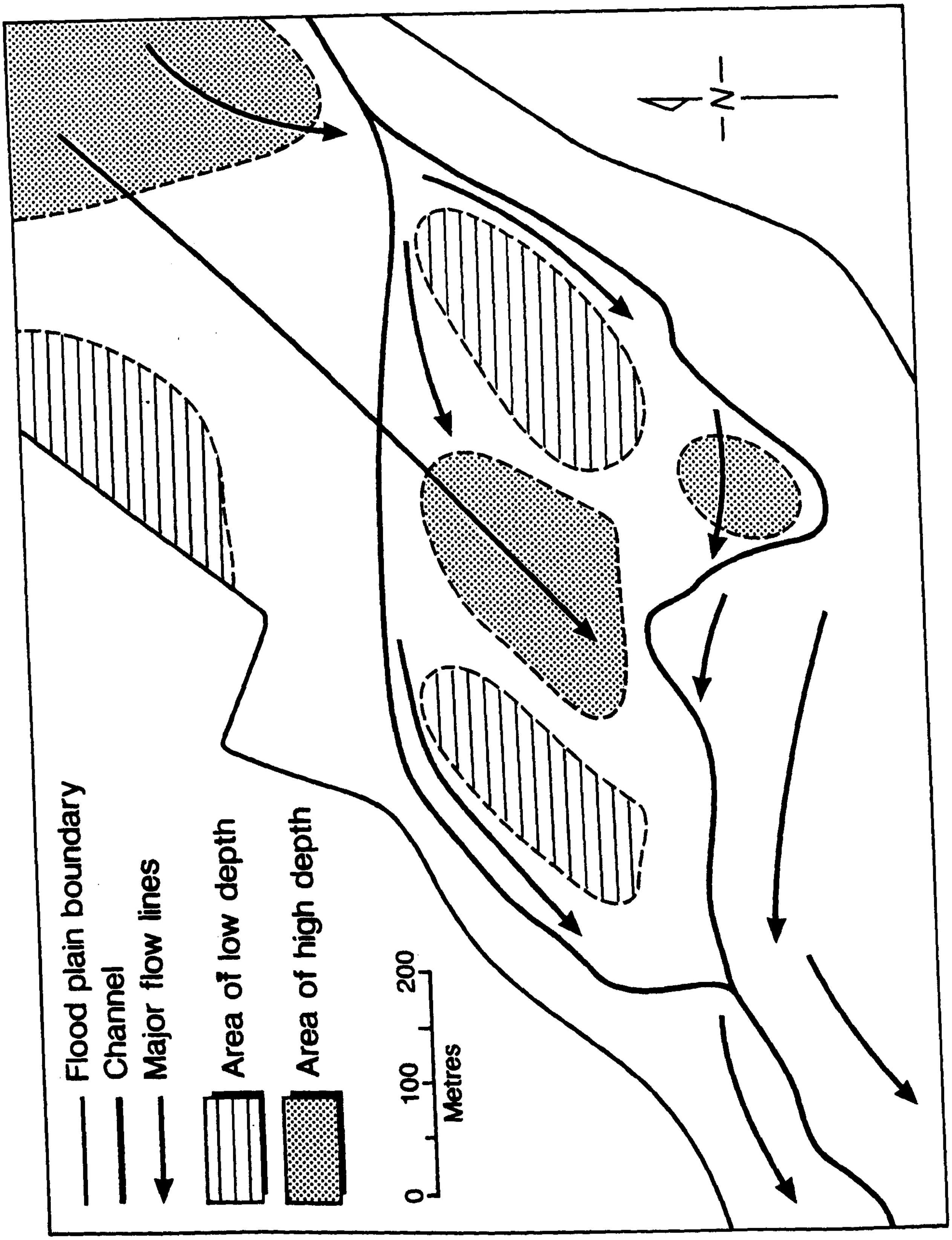


Figure 8.10: A summary of geomorphologically significant information derived from Figures 8.6 - 8.9.

floodplain depositional environment in these areas. Secondly, as such results are part of a dynamic simulation, it is possible to examine not only spatial but also temporal variations in the flow field to a high degree of resolution. Thus it is feasible to reconstruct detailed inundation sequences and the changes to the depositional environment of a particular floodplain area during the course of a flood event.

There is, however, no currently existing theoretical framework to link physically based models of floodplain hydrology to sediment deposition processes in the floodplain. Yet an understanding of floodplain deposition in terms of flow velocity and depth is essential if the medium to long term functioning of floodplains as sediment and contaminant stores is to be understood (see for example Beven and Carling, 1989). Floodplain sedimentation has been identified as a class of geomorphological problem where existing sediment transport theory does not apply. The dominant geomorphological process in such environments is the overbank **deposition** of fine material, often in the form of aggregates (Walling *et al.*, 1986; Walling and Bradley, 1989; Walling *et al.*, 1992). Existing theory, however, is primarily concerned with sediment **transport** of coarser sands and gravels and takes no account of sediment transport aggregation (see James, 1985; Lyn, 1987). The RMA-2 scheme is therefore in a unique position to contribute to a theoretical understanding of floodplain deposition as, not only can the model provide the required hydraulic data for theory development, but it has the ability to simulate every inundating event within a particular flow record. Such theory development would also require information concerning average sediment deposition rates over periods of 10 - 30 years at a high level of spatial resolution. Such data has, until recently, been impossible to document. However, the development of caesium^{137} dating techniques for floodplain sediments noted above overcomes this difficulty (see Figure 8.11) and now makes the development of a theory of floodplain depositional processes a realistic target for future research.

In relation to depositional processes within floodplain systems we can therefore conclude that two dimensional finite element models can:

i) provide a detailed and accurate reconstruction of flow data for complex floodplain micro-topographies at a scale commensurate within the known spatial variability of deposition,

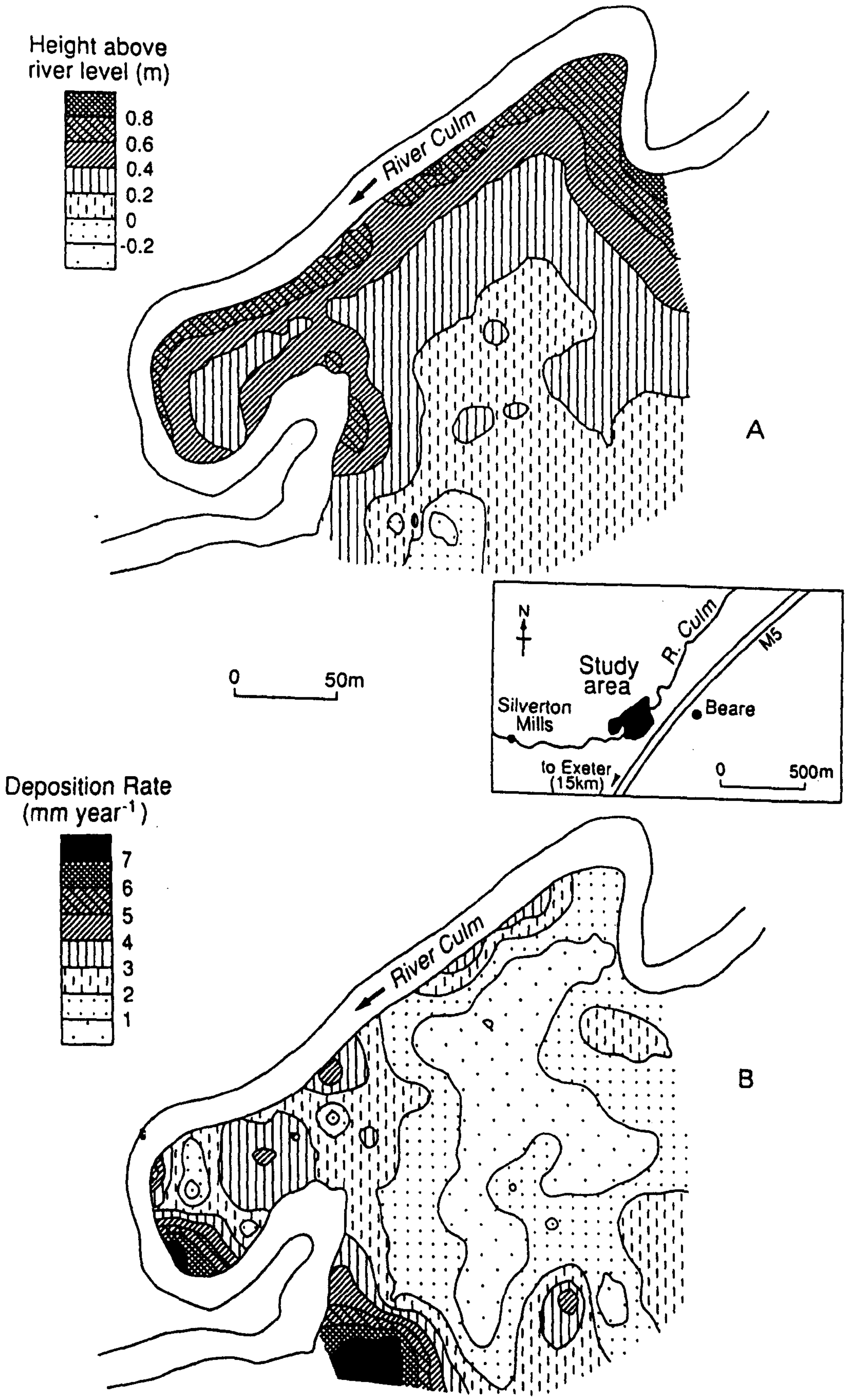


Figure 8.11: Floodplain deposition rates for a section of the River Culm derived from Cs^{137} inventories of floodplain sediment cores (taken from Walling *et al.*, 1992).

ii) be combined with data from caesium^{137} inventories of floodplain sediment cores to develop a better theoretical understanding of such processes.

8.3 The scope for the future integration of floodplain modelling into physical hydrology and fluvial geomorphology

Chapter 8 has provided independent confirmation that the spatially distributed predictions generated by RMA-2 are consistent with observed process behaviour. This appears to be due to the model's realistic representation of both floodplain topography and also the physical processes occurring within floodplain environments. Other distributed models in physical hydrology and fluvial geomorphology, such as the SHE (Abbott *et al.*, 1986), do not appear to have this capability (see for example Beven, 1989; Anderson and Burt, 1990). Yet, such a process representation would appear to be essential given that floodplain environments constitute one of the most dynamically active parts of the catchment, both hydrologically and geomorphologically. Considerable scope therefore exists for RMA-2 to be developed as a core modelling capability in these fields.

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